Installation Effect On And Estimation Of Ultimate Settlement Of Vertical Drain Treated Ground

A Thesis Submitted in Partial
Fulfilment of the Requirements
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Master of Technology
by

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CERTIFICATE

It is certified that the work contained in the thesis titled **"Installation Effect On And Estimation Of Ultimate Settlement Of Vertical Drain Treated Ground"**, by *Madhab Paul*, has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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ABSTRACT

Vertical drain installation effects on drain treated ground were studied conducting model tests. In the laboratory, soil samples were prepared using a reconsolidation technique. Soil samples were prepared in cylindrical metal tanks. Consolidation tests were carried out without drain (untreated soil) and with drain installation on the sedimented and consolidated soil samples. After consolidation, miniature cone penetration tests were carried out and water contents measured at different radial distances from the center of the sample and depths, in each test. The test results were analysed and discussed. Drain installation effects are studied comparing ultimate settlements, cone penetration resistances and water content values of the tests conducted. Ultimate settlements observed are more in the case of drain treated samples compared to those of untreated samples. It is observed that disturbace due to drain installation increases with the increasing size of shoe, OCR and aging.

An observational method is proposed to estimate long term settlement from observed time-settlement data. The proposed method is compared with the Asaoka's and hyperbolic methods for long term settlement estimation. The proposed method gives good estimation of long term settlement.

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MADHAB PAUL

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Notations

$a =$ width of the band shaped drain.

$a_0 = \dfrac{\beta_0}{\Delta t}$.

$a_1 = \dfrac{(\beta_1 - 1)}{\Delta t}$.

$a_s =$ a constant.

$b =$ thickness of the band shaped drain.

$b_s =$ a constant.

$C_c =$ compression index.

$C_s =$ swelling index.

$C_{vr} =$ coefficient of consolidation in the radial direction.

$C_{vz} =$ coefficient of consolidation in the vertical direction.

$c = \frac{2H^2}{3C_v}$, for double drainage,

$= \frac{H^2}{2C_v}$, for single drainage.

$D =$ constrained modulas

$D_e =$ equivalent diameter of drain influence zone.

$d_m =$ equivalent diameter of the mandrel cross section.

$d_s =$ outer diameter of smeared zone.

$d_w =$ equivalent diameter of drain.

$d_{15} =$ the particle diameter at which 15% of the particles are smaller in size.

$d_{50} =$ the particle diameter at which 50% of the particles are smaller in size.

$d_{85} =$ the particle diameter at which 85% of the particles are smaller in size.

$e =$ void-ratio

$G =$ specific gravity of the soil.

$H =$ length of drainage path.

$h =$ the distance along the drain between maximum and minimum pore pressure locations.

$k_h =$ Coefficient of horizontal permeability of the undisturbed soil.

k_s = Coefficient of horizontal permeability of the soil within the smear zone.

k_v = Coefficient of vertical permeability of the undisturbed soil.

k_w = permeability of filter jacket of the drain.

$k_{geotextile}$ = permeability of the geotextile.

k_{soil} = permeability of the soil.

LL = liquid limit.

l = length of the drain when drainage occur at one end only, and

l = half length of the drain when drainage occurs at both ends.

m_v = Coefficient of volume change.

N = an integer.

n = drain spacing ratio = $\frac{D_e}{d_w}$.

PL = plastic limit.

p = effective stress.

q_c = cone penetration resistance.

q_w = discharge capacity of the drain at hydraulic gradient of one.

$R(z,t)$ = the rate of loading.

r = radial distance.

S = drain spacing.

s = smear zone ratio =(d_s/d_w).

$s(t)$ = settlement at any time t.

s_n = settlement at time, t_n.

s_f = ultimate settlement.

T_r = time factor for radial flow.

T_v = time factor for the vertical flow.

t_{90} = time required for 90% degree of consolidation.

U = degree of consolidation for three-dimentional flow.

U_z = degree of consolidation for one-dimensional flow.

U_r = degree of consolidation for radial flow.

$\bar{u}$ = pore water pressure at any time, t.

w_c = water content.

z = vertical distance from the top.

z_1 = distance from the drainage end of the drain.

OCR = over consolidation ratio.

O_{15} = the opening size at which 15% of the openings in the geotextile are smaller.

O_{50} = the opening size at which 50% of the openings in the geotextile are smaller.

O_{95} = the opening size at which 95% of the openings in the geotextile are smaller.

σ_h = total lateral stress on the drain.

$\beta_n (n = 0,\ 1,\ 2,\ ...n)$ = unknown parameters.

γ_w = unit weight of water.

ν_s = Poisson's ratio of soil in the smear zone.

Δt= constant time interval between consecutive settlement.

Chapter 1

INTRODUCTION

As more and more land becomes subject to urban or indistrial development good construction sites are difficult to find. In recent years, an increasing need is felt for various types of construction in low land, coastal areas and other areas which are not suitable for stable foundations, because of one or more of the following reasons:

- Low strength (bearing capacity)
- High compressibility
- Low (or high) permeability
- Poor volume stability (shrinking and swelling)
- Susceptibility to liquefaction
- Detrimental physical or chemical changes, etc.

The aim of ground improvement is to make the poor soil strata suitable for sound foundations. Ground improvement techniques are also increasingly being applied in the rehabilitation of hazardous waste disposal areas. They also deal with the constructive use of high-energy waste materials such as slag and fly ash.

The basic principles of ground improvement techniques are not new; indeed, some of the techniques used today may be more than 2000 years old. However, significant advances have been made in recent times. Ground improvement techniques can be classified in the following four groups:

Hydraulic Modification: In-situ soil is consolidated by lowering ground water table through pumping from trenchs or boreholes in coarse grained soils while fine grained soils are improved by preloading with or without vertical drains.

Mechanical Modification: Soil is densified by application of short term external forces. Surface compaction is carried out by Static, Vibratory/Impact or Plate Rollers. Deep compaction is performed by Heavy Tamping at the surface or by Vibration at depth.

Physical & Chemical Modification: By mixing admixtures (like natural soils, cementing materials, bitumen, etc.) with the in-situ soil, engineering properties of in-situ soil are improved. Improving the engineering properties of soil by admixtures is often simply referred to as *soil stabilization*, particularly in road works. By thermal methods ground may also be improved. By heating, mineral structure of the soil can be altered to get better performance of the soil. Freezing solidifies part or all of the water and bonds individual particles together.

Modification by Inclusion & Confinement: Soil is reinforced by fibers, strips, bars, meshes and Geosynthetics. Reinforcing elements are used to construct stable earth-retaining structures. In-situ reinforcement is achieved by nails and anchors.

The choice of a particular method of ground improvement depends upon many factors including:

- Degree and type of improvement required
- Type of in-situ soil
- Construction time available
- Possible damage to the adjacent structures or pollution of ground water resources
- Techno-economical feasibility.

The desirability of a particular method of ground improvement is largely percieved in terms of environmental impact and energy consumption. However there may be situations in which a combination of two or more of the ground improvement techniques is required for the improvement of the in-situ ground, (Mitchell, 1981 and Hausmann, 1990).

Preconsolidation By Surcharge Load & Vertical Drain

To reduce post construction settlement and increase strength, soft clay may be preconsolidated using surcharge load. To preconsolidate clayey soil having low permeability,only by surcharge loading, large time is required. To decrease the time required for preconsolidation, both surcharge load is applied and vertical drains are installed. Vertical drains are artificially created drainage paths, which can have a

variety of physical characteristics and be installed by different methods. Due to the hydraulic gradient created by preloading, pore-water flows in a horizontal direction towards the drains and then along the drains vertically to the permeable drainage layer laid on top of the clay layer. Thus the vertical drains reduce the drainage path and thereby reduce the consolidation time. They also take the advantage of the higher horizontal permiability of the clay layer. By installing vertical drains one can reduce the time required for surcharge loading, thickness of surcharge fill and other consequent problems. These benifits are partially offset by the cost of the drain and the drainage blanket which is required to conduct water to outside the loaded area, and the cost of drain installation.

Vertical drains may be classified into three general categories:

- Sand drains
- Fabric encased sand drains
- Prefabricated strip drains.

In the design process, the drain capacity, drain installation effects on the consolidation of the clay layer, the total consolidation settlement likely to occur, etc. have to be estimated. The effect of installation of vertical drains on total consolidation settlement is to be considered. If the actual total settlement is more than the estimated total settlement, the constructed facility may not serve properly. Fig.1.1 shows a typical *vertical drain* installation procedure. Because highly efficient drain installation methods have been developed, many varities of prefabricated strip drains appeared in the market and the cost of the drain has decreased appreaciably; preloading with vertical drains has become an economic alternative to the installation of deep foundations or other methods of ground improvement.

This thesis consists of, in addition to Chapter-1, Introduction, following chapters:

Chapter-2 discusses the literature survey and previous research work relating to this subject. It mainly deals with the theory of consolidation and it's application to the design of vertical drains. Developments in vertical drains are presented. Evaluation of engineering properties of the ground to be modified by installing vertical drains along with preloading, characteristics of vertical drains, drain installation procedures and other associated problems have been discussed.

The properties of (i) soil used in Model Test and (ii) sand used to prepare sand wicks, experimental set-up and the procedures have been presented in Chapter-3. Test results are discused and analysed in Chapter-4. The effects of vertical drain installation on drain treated ground are studied. Chapter-5 is about the estimation of long term settlement from in-situ measurements. Obsevational methods to estimate ultimate settlement have been discussed and a modification is suggested.

Chapter-6 presents the conclusions arrived at from the research carried out.

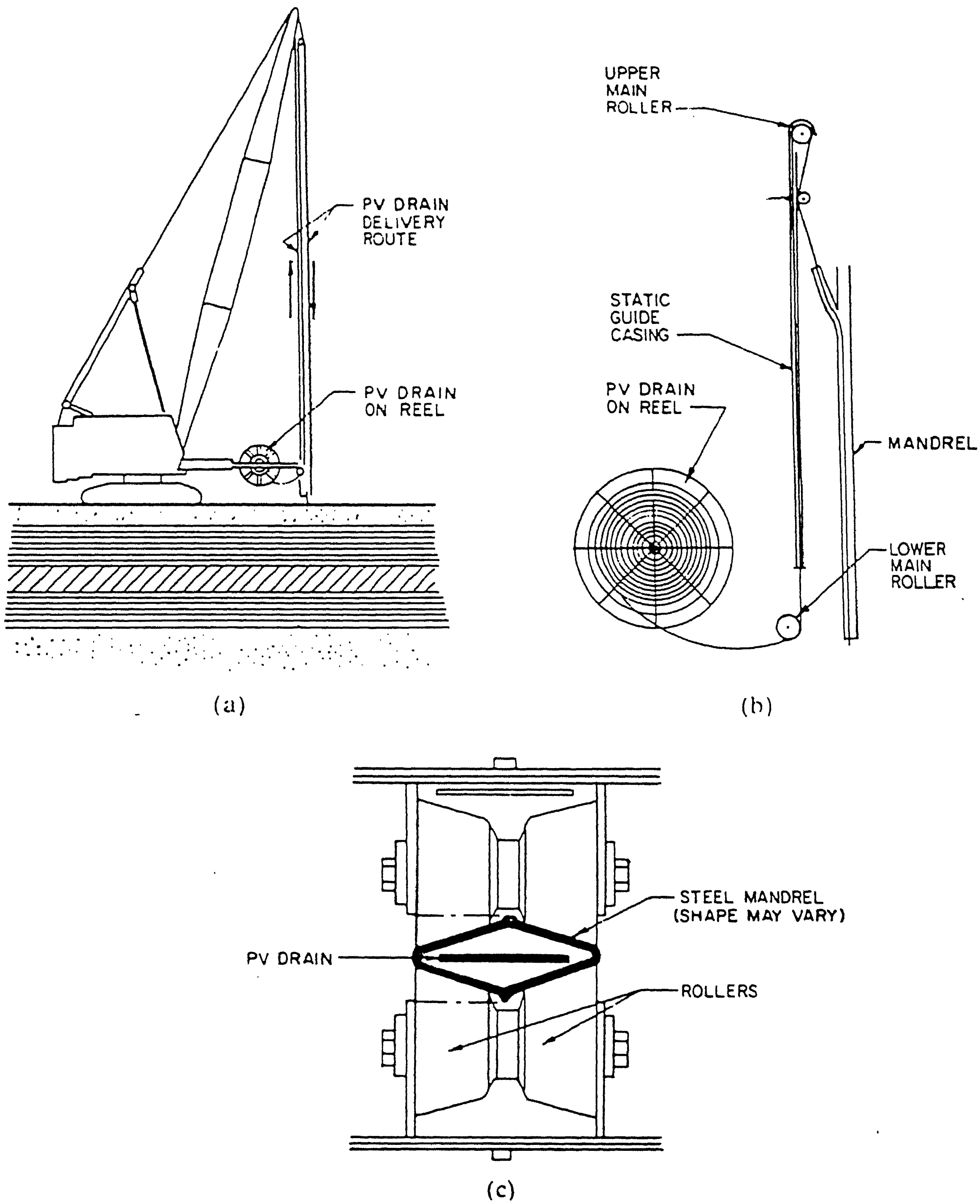


Figure 1.1: Installation of prefabricated vertical drains: (a) crane mounted installation rig, (b) drain delivery arrangement, and (c) cross section of mandrel and drain (after Rixner et al.,1986).

Chapter 2

LITERATURE REVIEW

2.1 Development Of Vertical Drains

The concept of vertical drains was developed in the late 1920's. First sand drain was developed. Presently, different types of vertical drains have been developed to improve the qualities of the vertical drains and ease of installation. Vertical drains may be classified into three general categories: sand drains, fabric encased sand drains (i.e, sand wicks) and prefabricated band drains. Prefabricated band drains may be classified into two groups: prefabricated plastic drains and prefabricated natural fibre drains.

2.1.1 Sand Drain

Moran proposed the use of *sand drains* in the late 1920's (Hansbo and Torstensson, 1977). Porter (1936) described some practical experiments on how vertical sand filled holes in a fine grained soil would affect the rate of settlement induced by external loading. Since then, sand drains have been increasingly utilised in order to shorten the time of consolidation of fine grained soils. A well is constructed by pushing a closed-end-mandrel into the ground or by water jet. The well is filled pouring sand into the well. Due to large settlement and shear failure continuity of sand drain may be threatened.

2.1.2 Sand Wicks

To facilitate construction, minimize sand waste and ensure continuity of the *sand drain*, the sand may be prepacked in a fabric sock, such as in *sand-wick* drain, which has a diameter of 65 mm (Dastidar et al. 1969), and then installed in the ground to

be treated by vertical drains. Installation procedure of sand wick is similar to that of the sand drain.

2.1.3 Prefabricated Plastic Drain

Kjellman (1930) developed the well known *Swedish card-board drain.* Geodrain, Alidrain and Ban-drain are commercial names of prefabricated drains having a grooved plastic core surrounded by a filter jacket. All these drains followed from *Swedish card-board drain.* The jacket material consists of non-woven polyester or polypropylene geotextiles or synthetic paper that function as a physical barrier separating the flow channel from the surrounding soft clayey soils and a filter to limit the passage of fine particles into the core to prevent clogging. The plastic core serves two vital functions namely: to support the filter jacket and to provide longitudinal flow paths along the drain even at large pressures. Installation of prefabricated plastic drain is very fast and causes only little disturbance (Choa et al., 1979). Also continuity of the drain is not threatened by large settlements.

2.1.4 Prefabricated Natural Fiber Drain

Vertical drains have also been developed using natural fibres viz. jute and coir. Mohan et al. (1972) developed *rope drains* using coir fibre. The coir fibre is woven in the form of a long strip of mat about 150 mm wide and 10 mm thick and then rolled to give a hollow cylindrical tube, which is flexible like a rope. The outer effective diameter of *rope drain* is about 70 mm and unlike in *sand drain* and *sand wick*, there is a contineous hollow space at it's centre longitudinally. Mohan et al. (1977) conducted laboratory experiments and field tests at Salt Lake in Calcutta, India, using *rope drains* made by rolling a coir strip and concluded that the *rope drains* have positive advantages over conventional *sand drains* and other types of vertical drains. The material used is highly permeable and the central hollow space ensures uninterupted passage for the drainage. The flexibility and strength of the rope drain safeguard against loss of continuty in the event of shear failure by excessive embankment loading. Closer spacing is possible because of easy driving and leads to increased efficiency of the process. The fabrication of *rope drains* and their installation are simple and quick.

Lee et al. (1989) developed a vertical drain using jute and coir. The general form of *fiber drain* developed is a rectangular strip 80 mm to 100 mm in width and 8 mm to 10 mm in thickness. Four coir strands of 3 mm to 6 mm in diameter obtained from coconut fibre are enveloped by two layers of jute burlap. Fibre drains were used in several projects involving marine and fluvial clays to speed up consolidation. Lee et al. (1989) presented laboratory and field test results. Based on their laboratory tests they concluded that the axial and filter permeabilities of *fiber drain* are more than 10^{-5} m/s for consolidation pressures up to 400 kN/m^2. A specimen of *fiber drain* was tested on

an Isotron mechine. At rupture (i.e, at 8.7 % strain) the peak tensile force recorded was 6.8 kN. High tensile force is benefitial in withstanding the installation stresses and high impact energy commonly applied in soil improvement projects. They presented three case studies. Based on their field observations they concluded that *fiber drains* have adequate discharge capacity, strength and functioned well for observation periods of over two years. Pandey and Majumder (1989) discussed manufacturing, testing and application of Jute as a Geotextile. They also observed that application of *phenol-formaldehyde resin* imparted not only water repellency but also microbial resistivity and prevented the degradation of the jute fabrics. Venkatappa Rao et al. (1994) conducted model tests to study performance of *natural fiber strip drain.* The load-settlement behaviour was monitored up to a surcharge pressure of 0.283 kg/cm^2. They also studied biodegradation of the *natural fiber strip drain.* From their study they concluded that the *natural fiber strip drain* is effective in consolidating highly plastic clays. It will not clog and the biodegradation is less in saturated clays compared to free draining sands.

Relatively low cost of manufacture using indegeneous material and local labour makes the *natural fiber drain* attractive in countries where jute and coir are abundant. It is also eco-friendly.

2.2 Developments Of Consolidation Equation

The theoretical concept of the consolidation process was developed by Terzaghi (1923). The design of vertical drains is based largely on the extention of Terzaghi theory of consolidation to cover the case of radial flow of water into a vertical well.The basic theory of vertical drain was presented by Rendaulic (1935) and Barron (1948) and later summarised by Richart (1959). Biot (1941) presented solution to the general three dimensional consolidation problem.

Barron (1948) presented the first exhaustive solution to the problem of consolidation of a soil cylinder containing a central sand drain. His theory was based on the simplifying assumptions of one-dimensional consolidation theory (Terzaghi, 1923). The following assumptions are usually made:

1. The soil is completely saturated,
2. Solid particles and the water are incompressible,
3. Darcy's law is valid and can be generalised for anisotropic medium and
4. The soil mass is homogeneous. Pressure increment Δp is applied instantaneously and that it is independent of time.

Barron's theory enables one to solve the problem of consolidation under two conditions, namely: (a) *free strain* — the vertical surface stress remains constant and the

surface displacements are non-uniform during the consolidation process; and (b) *equal strain* — the vertical surface stresses are non-uniform while the surface displacements are uniform during the consolidation process.

The consolidation equation can be expressed as:

$$\frac{\partial \bar{u}}{\partial t} = C_{vr}\left(\frac{\partial^2 \bar{u}}{\partial r^2} + \frac{1}{r}\frac{\partial \bar{u}}{\partial r}\right) + C_{vz}\frac{\partial^2 \bar{u}}{\partial z^2} + R(z,t) \tag{2.1}$$

where $\bar{u}$ = pore water pressure at any time t,
$R(z,t)$= the rate of loading; it is neglected when surcharge load is applied over a short period of time,
C_{vr} = coefficient of consolidation in the radial direction,
C_{vz} = coefficient of consolidation in the vertical direction,
r = radial distance,
and z = vertical distance.

2.2.1 Solution Of The Consolidation Equation

The general solution for consolidation by three dimensional flow for a given set of boundary conditions may become mathematically involved. However, the method of separation of variables can be applied to this problem, by considering Eq.2.1 to consist of two parts (neglecting the contribution of the rate of loading):

One dimensional flow:

$$\frac{\partial \bar{u}}{\partial t} = C_{vz}\frac{\partial^2 \bar{u}}{\partial z^2} \tag{2.2}$$

and radial flow:

$$\frac{\partial \bar{u}}{\partial t} = C_{vr}\left(\frac{\partial^2 \bar{u}}{\partial r^2} + \frac{1}{r}\frac{\partial \bar{u}}{\partial r}\right) \tag{2.3}$$

Carillo (1942) has shown that the solution of Eq.2.1 is given by a combination of the solutions for Eq.2.2 and Eq.2.3 as follows:

$$(1 - U) = (1 - U_z)(1 - U_r) \tag{2.4}$$

where U = degree of consolidation for three-dimensional flow,
U_z = degree of consolidation for one-dimensional flow,
and U_r = degree of consolidation for radial flow.

Solution of Eq.2.2 was obtained by Terzaghi (1923), as:

$$U_z = 100\left[1 - \frac{8}{\pi^2}\sum_{N=0}^{\infty}\frac{1}{(2N+1)^2}e^{-\frac{(2N+1)^2\pi^2}{4}T_v}\right] \tag{2.5}$$

where T_v = time factor for the vertical flow = $\dfrac{C_{vz}t}{H^2}$,
H = length of drainage path
and N is an integer.

Solution of Eq.2.3 (for radial flow), in the case of *equal vertical strain* under no smear and zero well resistance (Barron, 1948), is as follows:

$$U_r = 1 - e^{-(\frac{8T_r}{F(n)})} \tag{2.6}$$

where T_r = time factor for radial flow = $\dfrac{C_{vr}t}{D_e^2}$,
D_e = effective diameter of soil cylinder from which water will flow into the sand drain,
d_w = equivalent diameter of drain,
$F(n) = (\dfrac{n^2}{1-n})(\ln(n) - 0.75 + \dfrac{1}{n^2})$,
and $n = \dfrac{D_e}{d_w}$, is the spacing ratio.

Solution of Eq.2.3 (for radial flow), in the case of *free vertical strain* under no smear and zero well resistance (Barron. 1948), is as follows:

$$U_r = \left[1 - \sum_{\alpha_1,\alpha_2...}^{\alpha=\infty} \frac{4U_1^2(\alpha)}{\alpha^2(n^2-1)[n^2U_0^2(\alpha n) - U_1^2]} e^{4\alpha^2 n^2 T_r}\right] \tag{2.7}$$

where $U_1(\alpha) = J_1(\alpha)Y_0(\alpha) - Y_1(\alpha)J_0(\alpha)$

$U_0(n\alpha) = J_0(n\alpha)Y_0(\alpha) - Y_0(n\alpha)J_0(\alpha)$

$J_0, J_1 =$ Bessel functions of first kind, of zero order and first order, respectively

$Y_0, Y_1 =$ Bessel functions of second kind, of zero order and first order, respectively

$\alpha_1, \alpha_2, ... =$ Roots of the Bessel functions which satisfy:

$$J_1(\alpha n)Y_0(\alpha) - Y_1(\alpha n)J_0(\alpha) = 0$$

Richart (1959) has pointed out the fact that for n greater than 10, there is very little diference between the results obtained for average degree of consolidation by the two extreme conditions of *free strain case* and *equal strain case*. Fig.2.1 compares Barron's (1948) solutions for *equal strain* and *free strain cases* for drain spacing ratio, $n = 5$. This justifies the use of simpler equal strain solution.

With the advent of the use of computers, the finite element method (FEM), provides a suitable alternative to the use of the above conventional method in the design of vertical drains and in the prediction of their performance. The advantages basically lie in its applications, where the sub-soil condition is heterogeneous, caused either by disturbance during drain installation or by natural deposition. Its use is also more revelant when complex boundary conditions arise. Smith (1982) analysed different types of consolidation problems using FEM. Investigation of the performance of *sand-wick* in soft Bangkok clay by means of the FEM was carried out by Brenner and Prebaharan (1983). Zeng et al. (1987) presented an analysis of sand-drained ground by FEM and found FEM as a suitable method for this analysis provided the soil parameters used are realistic. Bergado et al. (1993) investigated the performance of prefabricated vertical *band-drain* both in the laboratory and in the field using FEM based on unit-cell concept. This is a simplified assumption whereby each drain works independently around a circular zone of influence, and all strains within the soil mass occur only in the vertical direction. They observed that settlement prediction is good when the ratios:

- $\left(\frac{coefficent\ of\ horizontal\ permeability,\ (k_h)}{coefficent\ of\ permeability\ in\ the\ smear\ zone,\ (k_s)}\right)$ and

- $\left(\frac{\textit{outer diameter of smear zone, } (d_s)}{\textit{equivalent diameter of drain, } (d_w)}\right)$

are assumed correctly. Xie et al. (1994) used FEM to compare the prediction versus performance of soft ground improved by vertical drains. They concluded that to achieve more realistic prediction the non-linear behaviour of vertical drain, such as *well-resistance* and the main deformation properties of the soft clay such as dilatancy, etc. should be taken into consideration and for this purpose FEM is suitable.

Karunaratne et al. (1989) presented a numerical model to analyse a narrow strip of compressible clay treated with vertical drains in a large soft clay deposit under a reclamation fill, by replacing the soil and vertical drains with a homogeneous soil of equivalent hydraulic conductivity:

$$k_v' = \frac{32}{\pi^2 \nu_s}\left(\frac{h}{D_e}\right)^2 k_h + k_v \tag{2.8}$$

where k_h and k_v are the permeabilities of the soil in the horizontal and vertical directions respectively,
$h =$ the distance along the drain between maximum and minimum pore pressure locations,
$D_e =$ equivalent diameter of drain influence zone,
$\nu_s =$ Poisson's ratio of soil in the smear zone.

The accuracy of settlement prediction depends on realistic moduli and hydraulic conductivity values of the soil used in the analysis.

Asaoka (1978) and Kodandaramaswamy and Narasimha Rao (1979) have presented observational methods to predict ultimate settlement due to consolidation. These will be discussed in Chapter - 5.

2.3 Equivalent Diameter Of Drain (d_w)

If the drain instead of being circular, is band shaped, Hansbo (1979) suggested that the equivalent diameter of drain (d_w) should be that of a cylinder having the same circumference, i.e,

$$d_w = \frac{2(a+b)}{\pi}$$

where a and b = width and thickness, respectively, of the band shaped drain (Fig.2.2). Subsequent finite element studies performed by Rixner et al. (1986) and supported by Hansbo (1987) suggested that the equivalent diameter for use in practice can be:

$$d_w = \frac{(a+b)}{2}$$

2.4 Drain Influence Zone (D_e)

The time to achieve a given percent consolidation is a function of the square of equivalent diameter of soil cylinder, D_e. This variable is controlable since it is a function of drain spacing(S) and pattern. Vertical drains are usually installed in square or triangular patterns (Fig.2.3). patterns. The relationships between the drain spacing (S) and the equivalent diameter (D_e) is given below:

1. Square pattern: D_e=1.128S

2. Triangular pattern: D_e=1.05S

2.5 Disturbance Effect

When vertical drains are installed, the structure of clay is damaged by way of smear and vibrations, particularly when vertical drains are installed using closed-end mandrel; consequently permeability of the soil adjacent to the drain decreases and the consolidation process is delayed. Barron (1948) extended the analysis of equal strain consolidation to account for the smear zone. He considered an annulus of smeared clay around the drain as shown in Fig.2.4. The analysis is based on the assumption that the clay in the smear zone will have one boundary with zero excess pore water pressure and the other boundary with an excess pore water pressure which will be time dependent. The area of the smeared zone is equivalent to the cross sectional area of the mandrel used (Casagrande and Poulos, 1969; Macdonald, 1985; Aboshi and Inoue,1986). Hansbo (1979,1981) modified the equations developed by Barron (1948) for prefabricated drain application. Within the annulus of smeared zone of outer diameter d_s, the remoulded soil has a lower co-efficient of permiability, k_s, than k_h, the horizontal co-efficient of permeability of the undisturbed clay. This leads to a new boundary condition between the undisturbed zone and the smeared zone. The factor F(n) now becomes:

$$F_s(n) = \ln(n/s) - 0.75 + (k_h/k_s)\ln s$$

where, s=smear zone ratio=(d_s/d_w)
The two parameters s=(d_s/d_w) and the ratio (k_h/k_s) are both difficult to estimate.

2.5.1 Shape & Size Of Smeared Zone

The disturbance is mostly dependent on the mandrel size and shape, soil macrofabric and drain installation procedure. Jamiolkowski et al. (1983) gave a relationship between the diameter of the smeared zone (d_s) and equivalent diameter of the mandrel cross section (d_m) as follows:

$$d_s = (2.5\ to\ 3)d_m$$

Hansbo (1987) recomended the following relationship between d_s and d_m based on the results of Holtz and Holm (1973) and Akagi (1979):

$$d_s = 2d_m$$

Bergado et al. (1993) have varified the above relationship by back calculations based on the observed time-settlement data of laboratory large scale consolidation tests. The influence of smear increases with increasing drain diameter for sand drains or mandrel diameter for prefabricated drains (Hansbo, 1981). Bergado et al. (1993) based on their back calculations also concluded that to decrease smear effect, mandrel cross section should be minimized (Fig.2.5). Onoue et al. (1991) proposed two zones of disturbance (zone-2 and zone-3) based on the measured void ratio data. Zone-2 is a zone partially remoulded and the drain installation causes a decrease in void ratio, and hence, permeability. Zone-3 is a fully remoulded zone. Zone-1 is unaffected by the drain installation. Fig.2.6 shows the zones of disturbance as suggested by Onoue et al. (1991).

Madhav et al. (1993) have modelled plastic drain treated soil as a two dimensional consolidation problem and considered the soil around the drain to consist of three zones, as suggested by Onoue et al. (1991). Based on their study they suggested that the response of plastic drain treated soil can be improved by reducing the mandrel size and modifying the shape of the strip drain to a circle or an ellipse.

2.5.2 Permeability Of Smeared Zone

Hansbo (1987) suggested that permeability of smear zone, k_s, can be taken as equal to k_v, the permeability of the soil in the vertical direction. Bergado et al. (1991) performed oedometer tests on samples taken shortly after drain installation on reconstituted soft Bangkok clay, at different distances from the drain. The results agree closely with the proposal of Hansbo (1987). Onoue et al. (1991) proposed two zones of disturbance with varying permeabilities (Fig.2.6)

2.5.3 Disturbance Effect On The Strength Of Soft Clay

Casagrande and Poulos (1969) concluded that driven sand drains (displacement type sand drain) are harmful in soft and sensitive clays due to disturbance in driving the drains. The strength and horizontal permeability of the soil are reduced. Akagi (1977) asserted that the installation of sand drain alone results in the consolidation of soft clays because of the large stresses induced during installation. High excess pore pressures are generated and after their subsequent dissipation, a gain in strength is achieved.

2.5.4 Effect Of Disturbance On Ultimate Settlement

Terzaghi and Peck (1967) discussed the effect of sample disturbance on '$e - \log p$' relationship. Due to disturbance soil structure gets disturbed. Madhav et al. (1995) observed that the destruction of the soil structure and pore water pressure induced causes the in-situ soil to undergo larger settlements of the ground compared to the untreated case. They analysed the additional settlements caused by remoulding due to drain installation effects and presented a simple model. They also compared the measured and / or estimated final settlements of untreated and drain treated grounds. They concluded that the settlement of drain treated ground is much larger than that of the untreated one, the ratio increasing with the degree of disturbance quantified by the pore pressures induced during installation.

2.6 Filter Capacity & Well-Resistance

Once water has entered into the drain, it is still possible for flow in the drain itself to be reduced. The problem is to determine the correct value of the drain discharge capacity, q_w, to be used in the design. This value is dependent on a number of factors such as volume of the core available for flow and the effect of the lateral earth pressure on that volume, folding and crimping of the drain due to large settlements, infiltration of the fine particles through the filter which by siltation could cause a reduction of flow capacity of the drain and durability of the drain system. Aboshi and Yoshikuni (1967) suggested that the parameter,

$$R = \frac{n^2-1}{n^2 F(n)}(k_h/k_w)(l/d_w)^2$$

could be used to take care of well-resisrance. In this case, the well-resistance effect is less than 10% when $R < 0.02$. Yoshikuni and Nakamodo (1974) used the parameter L, to represent the effect of well-resistance, where

$$L = \frac{32}{\pi^2}(k_h/k_w)(l/d_w)^2$$

Hansbo (1979) modified the equations developed by Barron (1948) for prefabricated drain applications. The modified general expression for average degree of consolidation (U_h) is given as:

$$U_h = 1 - \exp(\frac{-8T_h}{F}) \tag{2.9}$$

$$\text{where} \quad F = F(n) + F(s) + F(r) \tag{2.10}$$

where F is a factor which expresses the additive effect due to the spacing of the drains, F(n); smear effect, F(s) and well-resistance, F(r). Assuming that Darcy's law can be applied for flow along the vertical axis of the drain, Hansbo (1981) presented an expression for F(r) as given below:

$$F(r) = \pi.z_1(l - z_1)\frac{k_h}{q_w} \tag{2.11}$$

where, z_1 = distance from the drainage end of the drain,
l = length of the drain when drainage occur at one end only,
l = half length of the drain when drainage occurs at both ends,
k_h = co-efficient of permeability in the horizontal direction in the undisturbed soil,
q_w = discharge capacity of the drain at hydraulic gradient of one,
$k_w = permeabilityoffilterjacketofthedrain,$
d_w = is the equivalent diameter of drain,
and n = is the spacing ratio.

Holtz (1987), in a study of short term in-situ test of prefabricated drains concluded that well-resistance has little to no influence on the consolidation rate of prefabricated drain installations, even for long (> 50 m) drains. As long as q_w is greater than 100 to 150 m^3/year under the confining pressure acting on the drain, there should be no significant decrease in the consolidation rate. Drain bending, folding, kinking or wrinkling due to large settlements can possibly decrease the discharge capacity (Hansbo, 1983). Laboratory testing of drains in large cylinders of compressible clay has indicated that significant reductions in flow rates have occured after large settlements (15-30%), (Fellenius and Castonguay, 1985).

Tests on drains purposely bent showed that in no case was the flow completely cut off, although in some cases, the reduction was significant (Lawrence and Korner, 1988; Suits et al. , 1986)

2.6.1 Ramp Loading

Surcharge loading is not applied instantaneously, but is apllied gradually. Taylor (1948) first suggested an approximate method to account for consolidation during construction without drains He considered the linear ramp loading problem to be equivalent to that of an instantaneous load applied at the middle of the loading period. Kurma Rao et al. (1972) presented a design procedure for sand drains for time dependent loading. They combined the solutions presented by Schiffman (1959) and Lumb (1963) for vertical and radial drainages for time dependent loading for equal vertical strain condition, taking into consideration the variation of coefficient of consolidation due to vartical and radial flows. Yoshikuni and Nakanodo (1975) investigated the consolidation of clay cylinder by external drainage as an application of the theory based on consolidation potential. They concluded that the rate of consolidation of a clay cylinder by external radial flow depends upon the deformation condition and Poisson's ratio. They also observed that when the value of Poisson's ratio is nearly equal to 0.5, Terzaghi (1923) equation of consolidation is a close approximation to the actual state of consolidation. Oslon (1977) presented a mathematical solution for consolidation (both one-dimentional and radial) due to a single ramp load.

However, when using stage-loading procedures, the main problem is to evaluate the soil strength increase after each load increment (Ladd 1976, 1986).

2.7 Additional Width Requirement

There is a discrepancy between the predicted (i.e. designed) improvement and the observed field improvement due to vertical drain installation. This discrepancy is due to the following factors:

- Non-homogeneous ground, inaccurate soil parameters used.
- Discrepancy between assumptions in design theory and actual field conditions.

In the field generally vertical drains are installed only in a certain portion of the ground. But in the design procedure using Barron's solution it is assumed that drains are installed over the entire intensively loaded area. It is observed that near the boundary of the improved ground, predicted improvement does not occur. To overcome this problem the area in excess of the require treated area has to be treated. To determine the required area of installation of vertical drains in order to ensure complete improvement, a field sudy and theoritical calculations have been done by Kumamoto et al. (1988). They recomended to take the additional width (on both sides) to be improved as nearly equal to the layer thickness, to eliminate the time lag

in consolidation at the edge of the improved area. However the required additional width may be different in different field conditions.

2.8 Design Parameters

Design parameters required for the design and performance of pre-compression with vertical drains on soft cohesive deposits are:

- Horizontal and vertical coefficents of permeability (C_h and C_v).
- Stress history of the soil deposit.
- Extent of smear zone (d_s) due to installation of the vertical drain.
- Coefficient of horizontal permeability of the soil within the smear zone (k_s).
- Discharge capacity of the drain (q_w) and it's variation with total lateral stress(σ_h) and time (t).
- Transverse permeability of the filter sleeve.
- Effect of formation of kinks in the drain due to excessive settlement.
- Mechanical properties and durability of drain.

2.8.1 Evalution Of Soil Design Parameters

A comprehensive program of soil testing is required for the design of vertical drains. Because the usual priliminary sub-surface investigaion program is unlikely to provide sufficient information for a detailed design of a vertical drainage system, in-situ testing is necessary. Soil testing program should determine the following information:

- Macrofabric, geometry of drainage paths and the drainage boundary.
- Sand lenses in the clay layer.
- Consolidation and permiability characteristics in both horizontal and vertical directions.
- Stress-strain and strength characteristics.
- Stress history of the deposit.

For the prediction of the magnitude and rate of settlement, importance of the first three items is obvious. The stress-strain and strength properties are required because a stability analysis of the preloading fill must be performed. Depending on the strength of the foundation, stage construction may be required to avoid failure of the preloading embankment during construction. Both horizontal and vertical coefficents of consolidation (C_h & C_v) depend on the stress history of the deposit.

Evalution Of Permeability & Coefficient Of Consolidation

In-situ permeability test is to be carried out for this purpuse. Coefficient of consolidation may also be determined using piezometer probe (Jamiolkowski et al. , 1985). Falling Head Permeability Test, in general, gives lesser value of permeability compared to the Rising Head Permiability Test. In the case of Rising Head Parmiability Test, hydraulic fracture may occur. Hence Falling Head Permiability Test is preferred (Bhandari, 1979). The coefficent of horizontal consolidation can be evaluted by means of the approximate relationship:

$$C_h = \frac{k_h}{k_v} C_v.$$

Jamiolkowski et al. (1983) suggested that k_h value (determined by in-situ test) can be used with laboratory m_v value to evaluate C_h, using the relationship:

$$C_h = \frac{k_h}{m_v \gamma_w},$$

where γ_w is the unit weight of water.

Ratio Of Horizontal To Vertical Permeability

For soils with pronounced macrofabric, the ratio of $\frac{k_h}{k_v}$ can be very high, possibly 10. This high $\frac{k_h}{k_v}$ is benifitial, but may be reduced by the smear effect. Bergado et al. (1990) obtained:

- $\frac{k_h}{k_v}$=4–10,
- $\frac{C_{h,\, field}}{C_{v,\, lab.}}$=4

for soft Bangkok clay. A rough estimate of the in-situ anisotropy of the permiability of the clays, (k_h/k_v) can be made on the basis of the data given by Jamiolkowski et al. (1983).

Effect Of Preconsolidation Pressure

Hansbo and Torstenson (1977) based on their field study concluded that by preloading, preconsolidation pressure has to be exceeded to get the full benifits of the vertical drains. Nicholson and Jardine (1981) presented case studies at Queenborough bypass and compared consolidation parameters and settlements predicted from laboratory and in-situ tests with the field performance of the embankment with and without vertical drains. They found good agreement between these comparisons which confirm the predicted decrease in coefficient of consolidation as effective stress exceeded the preconsolidation pressure. Madhav and Miura (1994) presented a solution for one-dimentional consolidation of lightly OC clays. They extended the simple Terzaghi's theory for lightly over consolidated soils as a phase change process. They observed that lightly over consolidated soils in general exhibit faster rates of pore pressure dissipation and slightly higher rates of settlement than normally consolidated soils.

Presence Of Sand And Silt Layers

Presence of sand and silt layers enclosed in the clay layers has to be found out, if any. In case of stratified soils with closly spaced, continuous silt and sand layers, vertical drains may be of little value. Only preloading may be sufficient.

2.9 Drain Characteristics

2.9.1 Filter Characteristics For Sand Drain

The clay particles outside the sand drain should not clog the drain and at the same time discharge capacity of the drain should be adequate. Lambe and Whitman (1969) suggested:

1. $\frac{d_{15,\,drain}}{d_{85,\,clay\ layer}} < 4$ to 5, to prevent clogging.
2. $20 < \frac{d_{15,\,drain}}{d_{15,\,clay\ layer}} > 4$ to 5, to ensure 16 to 25 times greater permiability of sand drain compared to that of clay layer.

where d_{15} = the particle diameter at which 15% of the particles are smaller in size,

and d_{85} = the particle diameter at which 85% of the particles are smaller in size.

2.9.2 Filter Characteristics Of Prefabricated Drains

Important performance characteristics of prefabricated drains are the transverse and longitudinal permeabilities as well as their durability. These characteristics depend in the filter sleeve and core components of the drain. Mechanical properties are important for installation, and they may also play a role in the long term performance. Since almost all filter sleeves used today for prefabricated drains are non-woven geotextiles; geotextile filter concept, research results and available experience can be used to design and specify the materials for drain filter sleeves.

The requirements for an effective geotextile filter are:

- The filter must prevent piping of the adjacent soil.
- Soil particles already in suspension must be allowed to pass through the filter without clogging the filter.
- Head loss in the filter should be negligible.
- The filter characteristics must be maintained through out the life of the project.
- They must have sufficient tensile strength and resistance to tearing.

Christopher and Holtz (1985) deloped a design method for geotextile filter based on recomendations of Shober and Teindt (1979), Lawson (1982), Hoare (1982), Carroll (1983). Soil retention criterion is used to establish the largest allowable opening in the geotextile. For silts, clays, and non-woven geotextiles:

1. For steady flow:
   $O_{95} \leq 1.8d_{85}$ and $\leq 0.3mm$
2. For dynamic, pulsating or cyclic flow:
   $O_{50} \leq 0.5d_{85}$

In these relationships, O_{95} is the opening size at which 95% of the openings in the geotextile are smaller. For practical purposes, O_{95} is equal to the apparent opening size (AOS) expressed in mm, which is a measure of the largest hole in the geotextile. O_{50} is the opening size at which 50% of the openings are smaller, and d_{85} is as defined earlier.

Permeability criteria of prefabricated drains are related to the fabric porosity. The basic requirement is that the geotextile must be and remain more permeable than the adjacent soil. Christopher and Holtz (1985) suggest:

- For critical application and/or severe conditions:
  $k_{geotextile} \geq 10k_{soil}$

- For less critical, less severe situations:
  $k_{geotextile} \geq k_{soil}$

In typical soil conditions where prefabricated drains would be used, almost all geotextiles have sufficient permeability. An additional criterion for prefabricated drain filters is that their permittivity (flow rate per unit area under a given head) must be at least as great as the overall flow requirements of the system (Holtz, 1987).

Poorly graded soils, such as gap graded soils, have been identified as problem materials as far as filter protection is concerned, because seepage water may bring with it enough soil particles to clog the filter, thus making it less permeable. Clogging resistance criterion depends on the critical nature of the project and on the severity of the hydraulic loading conditions. According to Holtz (1987), geotextile filter with the largest opening size based on soil retention criteria should be specified for less critical/less severe applications, which are common in many vertical drainage situations. It is also recomended that in potential clogging situations such as gap-graded and silty soils, the specification should include a porosity qualifier for non-woven geotextiles:

1. porosity $\geq 30\%$, is a must.
2. $O_{95} \geq 3d_{15}$
3. $O_{15} \geq 3d_{15}$

Where O_{15} = the opening size at which 15% of the openings in the geotextile are smaller. These specifications ensure that the finer soil particles can pass through the filter without clogging it. For critical applications and severe conditions, the geotextile filter should be selected to meet all the retention and permeability criteria given for less critical and less severe conditions. In addition, soil-geotextile filtration tests using representative samples of the soils at the site should be conducted, (Holtz, 1987).

Gradient Ratio (GR) test is generally preferred for soils with permeabilities greater than 10^{-5} cm/s (silts, clayey and silty sands), Carroll (1983). Laboratory permeameter tests on specific soil-geotextile systems, such as those developed by the U.S.Corps of Engineers, allow measurement of the gradient in the soil (after the flow has stabilized) is termed Gradient Ratio (GR). The clogging criteria (Carroll, 1983) proposed is that the GR should be less than 3, in such a filtration test in order to ensure satisfactory performance in the field. Interpolation of long term filter performance data is complicated by the fact that, the textile may undergo chemical degradation and clogging due to biological growth rather than perticle movement. So, for critical projects, field evaluation is necessary. In the evaluation of geotextile filter, deterioration of the filter by biological or chemical attack should be considered (Christopher and Holtz, 1985).

2.9.3 Mechanical Properties Of Core And Filter Of Prefabricated Drains

Prefabricated vertical drain consists of a synthetic filter jacket surrounding a plastic core. The filter jacket and plastic core should have sufficient strength to withstand installation stress. Mechanical strength characteristics, especially the tensile strength of the core and filter, are important, because the filter is likely to be subjected to high tensile stresses during it's installation. Holtz (1987) suggested that samples of the entire drain (10.0 to 35.0 m long) should be pulled at a constant but rather rapid rate of tension (10 to 15% per minute) to simulate stress-strain conditions during installation. The drain should have a minimum tensile strength of 5 kN/m width and the strain at maximum tensile stress should be between 2% and 10%.

2.10 Relative Effects Of Key Parameters

Rixner et al. (1986) investigated the relative effects of the key parameters: horizontal coefficient of permeability (k_h), equivalent diameter (D_e), equivelent drain diameter (d_w), coefficient of horizontal permeability in the smear zone (k_s), on the time (t) required to achieve a particular degree of consolidation. Accordingly the greatest potential effect on consolidation time (t) is due to the variations of C_h and D_e. The value of C_h, which can easily vary by a factor of 10, has the most dominant effect on 't'. The effects of the coefficient of permeability and diameter of the smear zone can also be significant. The equivelent diameter of drain has only a minimal influence on consolidation time (t).

2.11 Selection Of Drainage System

Sand drains are installed with the aid of either a closed-end-mandrel or an open-end-mandrel with a jetting device. When closed-end-mandrel is used, the displacements induced by driving of the drain often cause severe disturbance in the soil mass. Consequently both soil strength and coefficient of conslidation decrease. Jetted sand drain installation using open-end-mandrel (non-displacement method) is better but more expensive and also water is not easily available always. Non-displacement method is also laborious and time consuming, especially when drains of lengths 20 m to 40 m or more are involved. Again disposal of spoil from the bore holes is an additional problem (Choa et al. , 1979). Sand with suitable grain size distribution is not always available. Continuity of drains is threatened by large settlements. The problem with sand drain can be eliminated using prefabricated band drain. Installation of prefabricated band shaped drains is also very fast. For the construction of the second airport project, Republic Of Singapore, prefabricated drains were installed to depths

upto 43 m and an average production rate of 2200 m drain per rig per 10 hour day was achieved (Choa et al., 1979). Installation of prefabricated band drains causes only a little disturbance. Therefore in-situ C_h value does not change much due to the installation. Also the continuity of the drain is not threatend by large settlements. Performance of prefabricated drains may be reduced due to clogging, increasing confining pressure, kinking or buckling. Where jute and coir are abundant prefabricated drains made of jute and coir may be used economically.

2.12 Scope Of The Thesis

To design vertical drains factors to be considered are: engineering properties and preconsolidation pressure of the ground, dimension of the drain, drain spacing and arrangement, drain installation effect on the drain treated ground, etc. When vertical drains are installed, structure and properties of the ground near the drain get altered and pore pressure is induced by the disturbance due to drain installation. Due to this disturbance, the ultimate settlement of the drain treated ground becomes larger than the untreated ground for the same load increment.

In the present investigation, installation effect on the vertical drain treated ground has been studied. Model tests were carried out to study how disturbance effect changes with change in size, preconsolidation pressure and aging. Estimation of long term settlement is also very important to control total and differential settlement of the structure to be constructed. It is very difficult to determine in-situ properties of the ground. The procedures to estimate long term settlement from observed time-settlement data are reported by Asaoka (1978) and Kodandaramaswamy and Narasimha Rao (1980). A new observational method to estimate long term settlement is proposed modifying Asaoka's method. All the three methods have been discussd and compared using time-settlement data published by several investigators for a number of field cases.

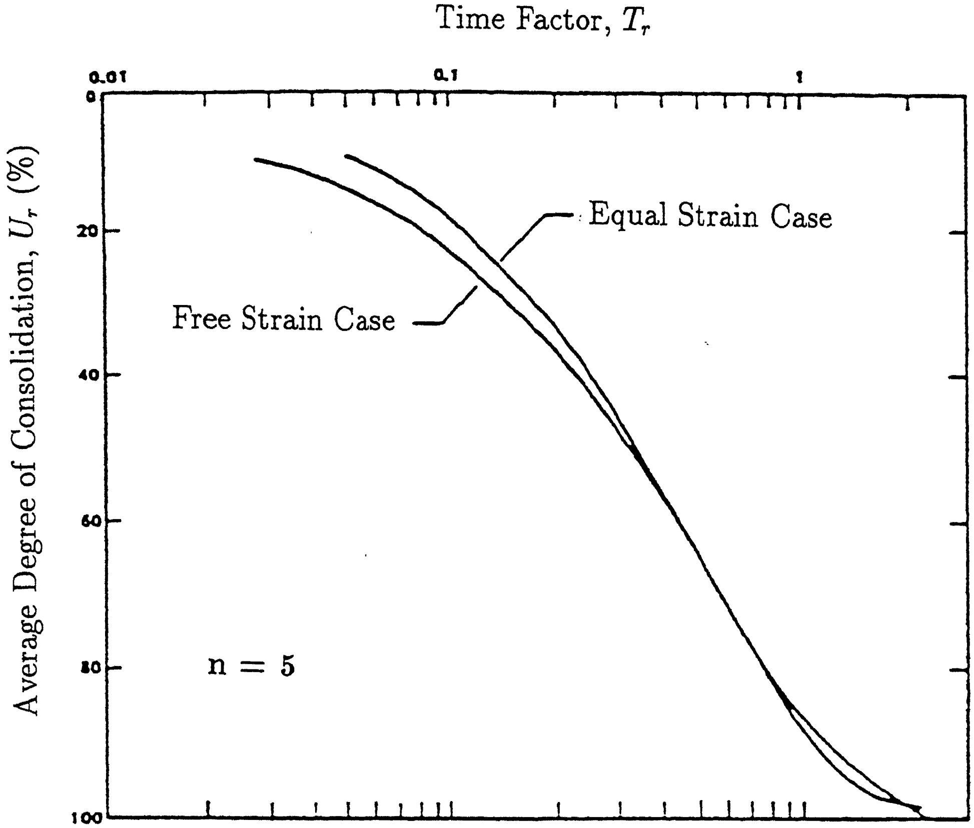


Figure 2.1: Average degree of consolidation versus time factor for free strain and equal strain boundary conditions: radial inflow tests with the drain spacing ratio = 5 (after Trautwein, 1980).

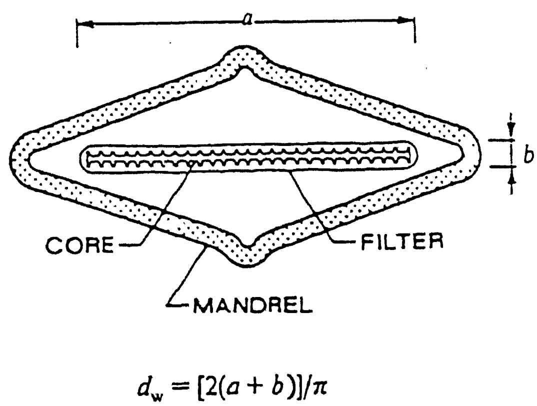


Figure 2.2: Equivalent diameter of a band-shaped drain (after Hansbo, 1979).

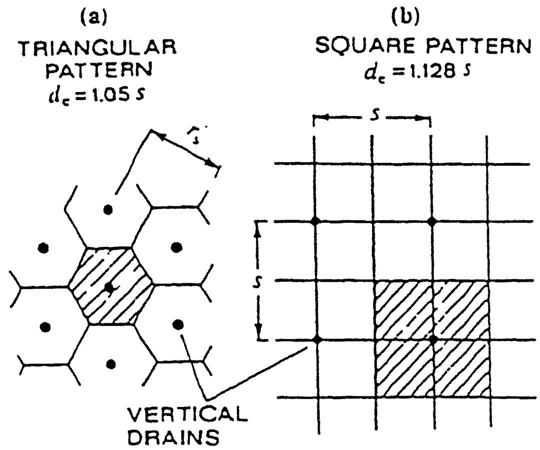


Figure 2.3: Different drain patterns: equivalent diameter.

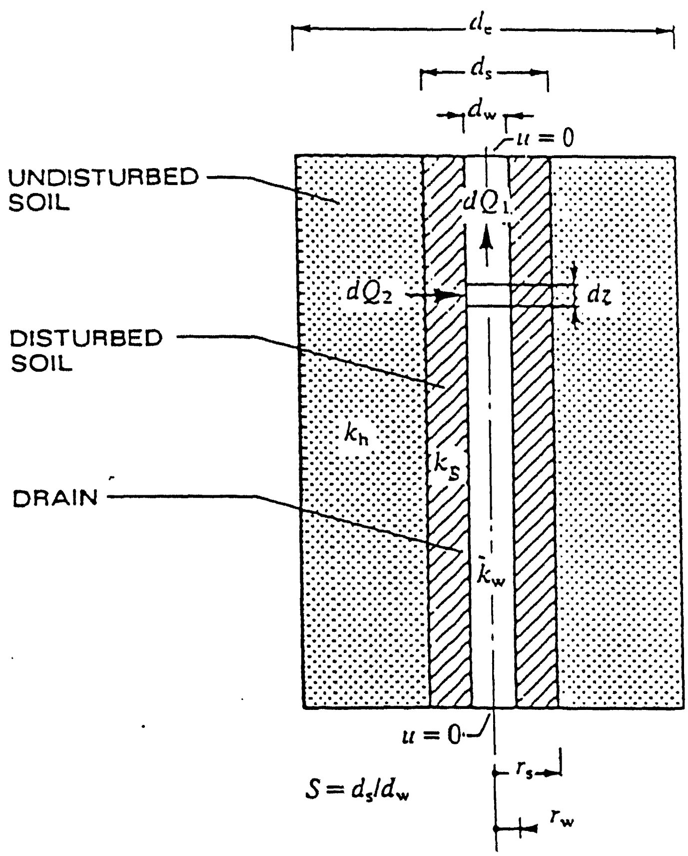


Figure 2.4: Section of the equivalent cylinder (after Barron, 1948).

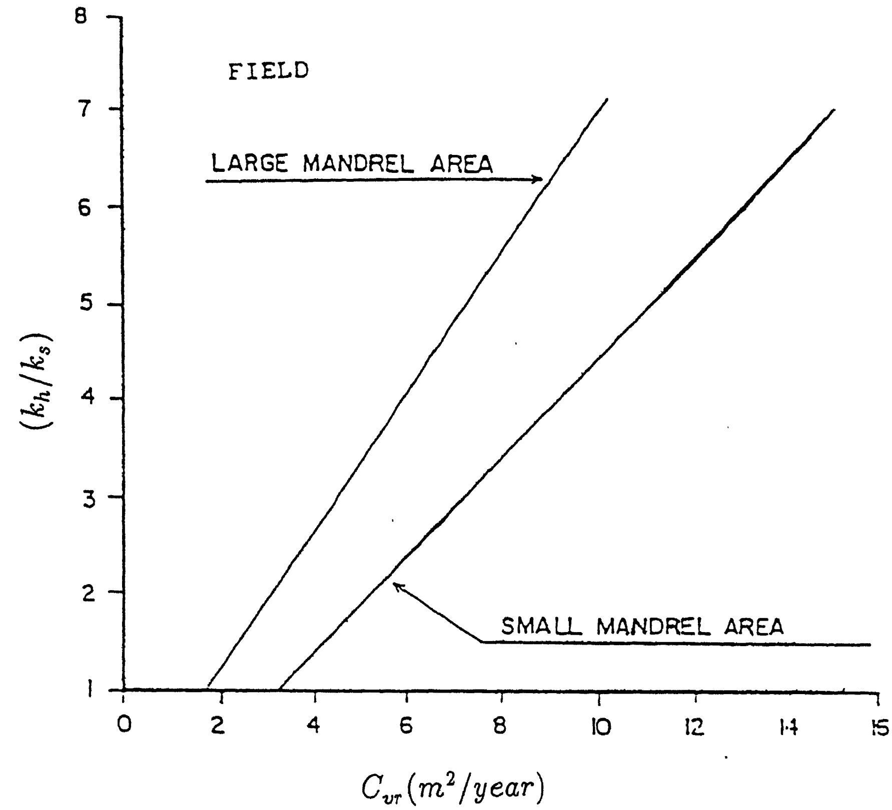


Figure 2.5: Back calculated sets of k_h/k_s and C_{vr} values (after Bergado et al., 1993)

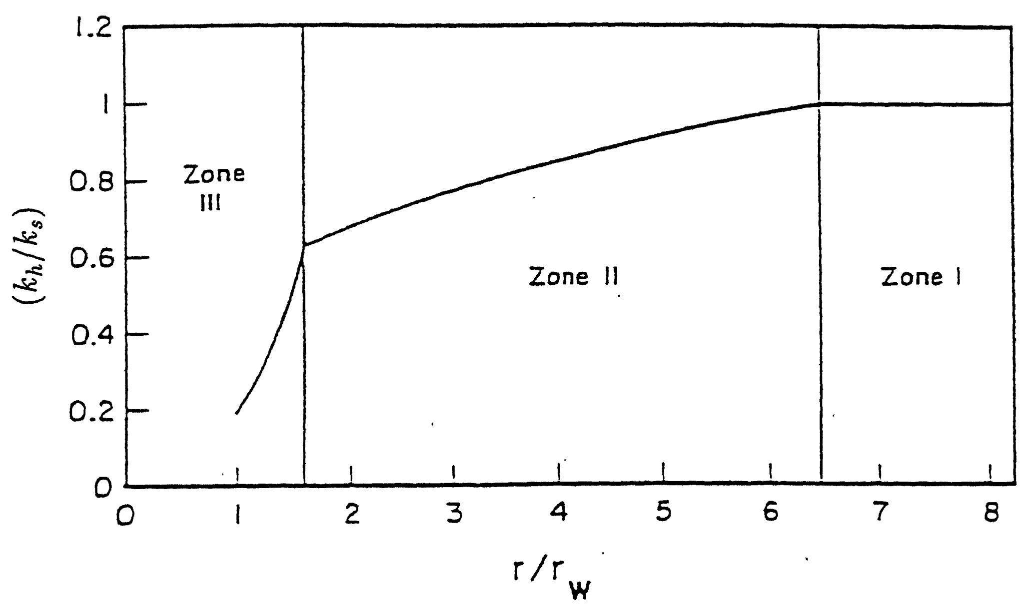


Figure 2.6: Variation of permiability with radious (after Onoue et al., 1991).

Chapter 3

MATERIAL PROPERTIES AND EXPERIMENTAL PROCEDURES

3.1 General

Model tests were carried out to study the vertical drain installation effect on settlement of drain treated ground. The properties of the silty clay and sand used in the model tests were determined in the laboratory. Plasticity characteristics, grain size distribution and engineering properties (LL, PL, specific gravity, coefficient of consolidation, compression and swelling indices, permeability, constrained modulus, etc.) of the silty clay used to prepare sedimented samples and permeability of sand used as drainage material, were determined. These material properties are presented in the following sections. Experimental set-up and test procedures are also described.

3.2 Material Properties

In the tests silty clay from *I.I.T. Kanpur Campus* was used to prepare soil slurry for the model tests. *Kalpi sand* was used to prepare sand wicks.

3.2.1 Properties Of Silty Clay

Grain size analysis, LL & PL tests, specific gravity and oedometer tests have been carried out. Sieve analysis and hydrometer tests were done to determine the grain size distribution. Liquid limit (LL) was determined by: (a) Casagrande method and (b) Penetration test.

Grain Size Analysis

Both sieve analysis and hydrometer tests were carried out. Fig.3.1 shows the result for the obtained grain size analysis. The soil consists of :

$$\text{Clay} = 31\ \%$$
$$\text{Silt} = 55\ \%$$
$$\text{Sand} = 14\ \%$$

LL, PL and Specific Gravity

Liquid limit has been determined using the mechanical liquid limit device (Casagrande type) and by penetration test (IS: 2720 – V). The results obtained are summarsied below:

$$(LL)_{Casagrande} = 36\ \%$$
$$(LL)_{cone\ penetration} = 41.3\ \%$$
$$\text{Plastic limit (PL)} = 16\ \%$$
$$\text{Specific gravity (G)} = 2.63$$

Oedometer Test Results

Two samples were prepared at a water content of 43 % (slightly higher than the liquid limit). Oedometer test was carried out with the usual loading intensities: 10, 20, 50, 100, 200, 400 and 800 kPa. When the change in dial gauge reading becomes negligible, the next increment of load is applied. The interval between two consequtive load increments was at least four hours. Unloading was also done in a similar fashion. The results obtained in the two tests are similar. Oedometer test results are presented in Fig.s 3.2, 3.3, 3.4 and 3.5. Both loading and reloading curves in (e, log(p)) axes are nearly straight lines (Fig.3.2). The values of the compression (C_c) and swelling indices (C_s) are 0.233 and 0.022 respectively. Fig.3.3 shows the variation of coefficient of consolidation (C_{vz}) with log(p). It can be noted that coefficient of consolidation (C_{vz}) varies from $2.573 \times 10^{-4}\ cm^2/s$ to $11.207 \times 10^{-4}\ cm^2/s$ and increases with increasing load intensity (p). Permeability (k) decreases with increasing load intisity and varies from 135.854×10^{-6} cm/s to 10.086×10^{-6} cm/s (Fig.3.4). Fig.3.5 represents relationship between constrained modulus (D) and effective stress (p). Initially i.e, upto 300 kPa, '$D - p$' plot is a straight line.

3.2.2 Properties Of Sand Used

Clay and silt fractions of Kalpi sand were removed by seiving. The fraction of Kalpi sand passing through 4.75 mm seive and retained on 0.075 mm seive was used in the preparation of sand wicks. Finer particles were removed so as to avoid clogging of the drain. Constant head permeability test was carried out to determine the permeability (k) of Kalpi sand. Samples were prepared at the expected density (1.72 gm/cm^3) of the sand in sand wicks. The test results are given below:

Test-1: k=2.55 $\times 10^{-3}$ cm/s.
Test-2: k=2.50 $\times 10^{-3}$ cm/s.
Test-3: k=2.48 $\times 10^{-3}$ cm/s.

The average value of the permeability is 2.51 $\times 10^{-3}$ cm/s which is nearly 19 to 42 times the permeability of the silty clay in the stress range 0.0 kPa to 2.6 kPa.

3.3 Model Test

Model tests were done in cylindrical metal tanks. Sedimented soil samples were prepared in the metal tanks. Consolidation tests were carried out: (a) on sedimented samples and (b) with drain installation on the sedimented and consolidated soil samples. After consolidation, miniatute cone penetration test was carried out and water contents measured at different radial distances from the center and depths in each test. Fig.3.6 shows the experemental set-up. Loads were applied on the samples by dead weights.

3.3.1 Model Dimension

Dimension Of Tank

Tank diameter = 27 cm,
Tank height = 65 cm

and at the bottom a valve is provided to allow drainage (Fig.3.7).

Shape And Size Of Drain

Two types of sandwicks were used:

- Circular sandwicks of diameter 25 mm and
- Rectangular sandwicks of size 30 mm X 9 mm.

Shape And Size Of Shoe

Three types of shoes (Fig.3.8) were used to install vertical sandwicks:

- Conical shoe of base diameter 30 mm and vertex angle of 60°.
- Conical shoe of base diameter 40 mm and vertex angle of 60°.
- Rectangular tapered shoe of base size 48 mm X 15 mm.

Circular and rectangular sandwicks were installed with the conical shoes and rectangular shoe, respectively.

3.3.2 Preparation Of Sedimented Soil Samples

Soil samples have been prepared using reconsolidation technique. McManus and Kulhawy (1993) describe the detailed methodology of the preparation of large size laboratory deposits of cohesive soils. Reconsolidation technique is used to produce uniform and nearly identical soil deposits in the laboratory.

Principle Of Sample Preparation

Soil slurry is prepared and poured into a tank above a filter layer and consolidated. The consolidation pressure is applied in stages to avoid squeezing of the soil slurry.

Slurry Preparation

Silty-clayey soil was air dried, broken down to powder form and sieved on 2 mm sieve. The portion of the soil passing through the 2 mm sieve was used in the preparation of slurry. Required quantity of soil was mixed with known amount of water and the mixture was kept 24 hours for saturation. In the present experimental study, slurry was prepared at the initial water content varying between 42.9 % and 43.3 %, slightly higher than the liquid limit of the soil. In each test the following steps were followed to control the water content of the slurry:

- Water content of the air dried sample was measured.

- For a given quantity of air dried sample, the balance amount of water to be added to get $w_c = 43\%$, was calculated.

- The calculated amounted of water was well mixed with the air dried sample.

70 mm thick layer of Kalpi sand was placed at the bottom of the tank and saturated. The excess water was allowed to drain through the bottom valve. In order to avoid intrusion of clay into the sand, a filter paper was placed over the sand layer. Oil was applied on the inner surface of the tank to reduce side friction. Side filter drains in the form of strips were placed along the inner surface of the tank. Slurry was well mixed again before pouring into the tank and water content of the slurry was measured. Slurry was poured into the tank, in three layers each 15 cm thick. After pouring first layer of slurry, the top surface was levelled off and two layers of filter paper in the form of sectors were placed. This procedure was repeated for the second layer and third layer. Then the sample was left for 24 hours to consolidate under it's own weight. Then, on the filter paper a layer of saturated sand of 50 mm thickness and a metalic plate having diameter 265 mm were placed. Load was applied on to the metal plate by dead weights.

Sedimentation

Soil slurry was consolidated at two different consolidation pressures:

- **3.25 kPa** applied in two stages: 0.0 kPa to 1.625kPa and 1.625 kPa to 3.25 kPa.

- **6.50 kPa** applied in three stages: 0.0 kPa to 1.625 kPa, 1.625 kPa to 3.25 kPa and 3.25 kPa to 6.50 kPa.

Dial gauge readings were recorded at 4, 9, 16, 25, 36, 49, 64, 90, 120, 180, 300, 480, 720 minutes and at every 12 hours, till the change in dial gauge reading becomes less than one divisions in one hour, during loading for each load increment. To study the effect of aging, after consolidating the soil sample at 6.50 kPa the load was sustained for another 10 days. After loading the samples were unloaded as:

- **Sample Consolidated to 3.25 kPa:** 3.25 kPa to 1.625 kPa and then 1.625 kPa to 0.0 kPa.

- **Sample Consolidated to 6.50 kPa:** 6.50 kPa to 1.625 kPa and then 1.625 kPa to 0.0 kPa.

For each stage of unloading dial gauge readings were observed and when change in dial gauge reading became very small (less than one division in one hour), next stage

unloading was done. To measure settlement (or swelling) three dial gauges at an angle of 120° with each other, were installed in each test. Average of the three readings was taken as settlement reading.

3.3.3 Tests On Untreated Soil

Slurry has been prepared as described earlier. Four tests were done for four different loading conditions:

(1) After preparation of slurry (Section 3.2.2), loading was done upto 26.0 kPa in stages: 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. Dial gauge readings were observed as stated earlier. After loading is complete, unloading was done in the following sequence:

26.0 kPa to 13.0 kPa, 13.0 kPa to 6.50 kPa, 6.50 kPa to 3.25 kPa,
3.25 kPa to 1.625 kPa and 1.625 kPa to 0.0 kPa.

(2) Reloading and again unloading were done after consolidating the slurry (Section 3.2.2) at 3.25 kPa. During reloading stresses successively applied are 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. Unloading was carried out at a rate of one fourth of the original rate after completion of reloading.

(3) Reloading and again unloading were done after consolidating the slurry (Section 3.2.2) at 6.50 kPa. During reloading stresses successibly applied are 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. Unloading was carried out at a rate of one fourth of the original rate after completion of reloading.

(4) Soil slurry was consolidated at 6.50 kPa and the load was sustained for another 10 days. Then unloading and reloading were done. Stresses successively applied during reloading are 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. After completion of reloading unloading was carried out at a rate of one fourth of the original rate.

3.3.4 Tests With Drains

Three types of samples were prepared (Section 3.2.2) for testing (after drain installations):

- Preconsolidated at a preconsolidation pressure of 3.25 kPa.
- Preconsolidated at a preconsolidation pressure of 6.50 kPa.
- Preconsolidated at a preconsolidation pressure of 6.50 kPa and aged for 10 days.

On each of these samples, three tests were done installing a central sandwick:

- Test (a): sand wick installed by a conical shoe of base diameter 30 mm and vertex angle of 60°.
- Test (b): sand wick installed by a conical shoe of base diameter 40 mm and vertex angle of 60°.
- Test (c): sand wick installed by a rectangular taper shoe of base size 48 mm X 15 mm.

Drain Installation Procedure

Sandwick was made pouring dry sand into a cotton sock. Sand was poured into the sock from a height of one metre. Sand wick was saturated with water before installation. Saturated sand wick was placed centrally above the shoe and using a steel rod of diameter 8 mm *shoe and sandwick* were forced driven centrally into the sedimented sample (Fig.3.9). After driving the sand wick the steel rod was slowly withdrawn.

Testing

After drain installation, in each test, the stresses successively applied were: 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. At each stress increment dial gauge readings were recorded at 4, 9, 16, 25, 36, 49, 64, 90, 120, 180, 240, 300, 480, 720, 960 minutes and at every 8 hours, till the changes in dial gauge readings were less than one division in one hour. Unloading was carried out at a rate of one fourth of the original load.

3.3.5 Cone Penetration Test & Water Content Determination

Miniature cone penetration test was done and water content was determined after each test, with or without drain, is over:

- at radial distances of 2.5 cm, 7.5 cm and 12.0 cm from center.
- at depths of 0.0 cm, 6.5 cm, 13 cm, 19.5 cm, 26 cm, 32 cm and 37 cm.

Fig.3.10 shows the sequence of cone penetration tests carried out on each sample. The miniature cone penetrometer (Fig.3.11) consists of a cone with base diameter 20

mm and vertex angle 60°, driving arrangements and load indicator. It is driven into the soil by pushing it manually and the force applied is noted from the load indicator. The force required to push the cone into the soil at the desired level is noted. The cone penetration resistance (q_c) is the force divided by the base area of the cone.

The results obtained from the experimental studies are discussed and analysed in the next Chapter.

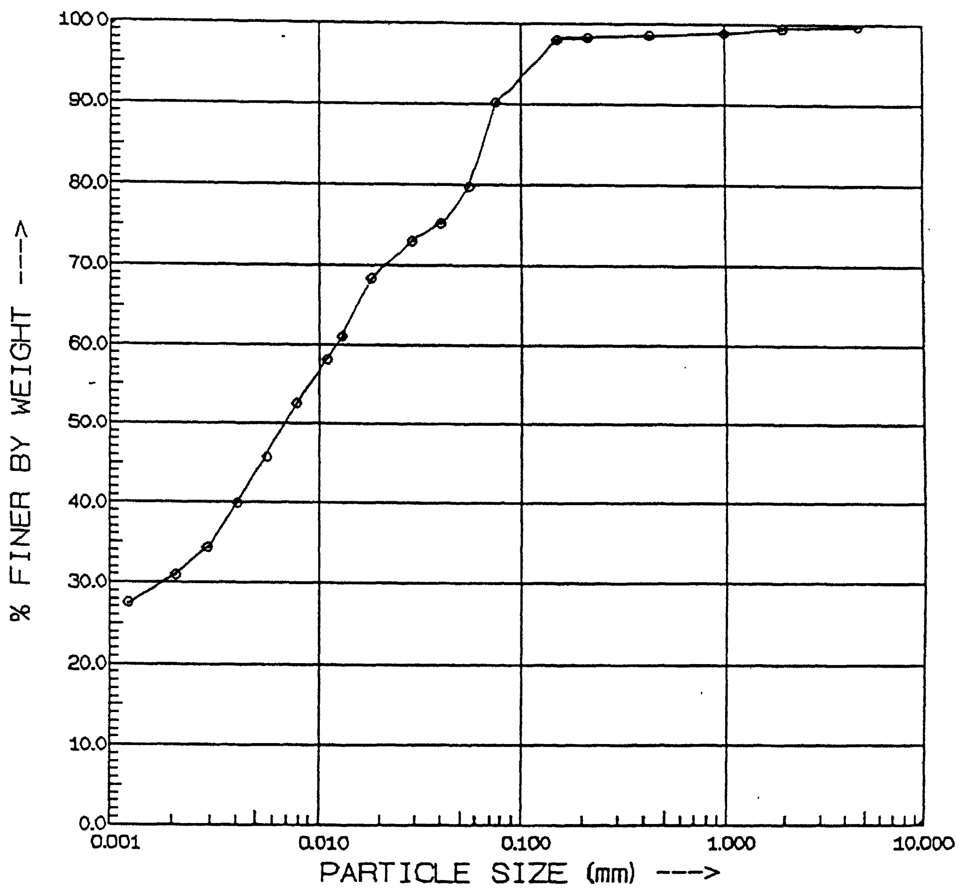


Figure 3.1: Grain size distribution curve for silty clay.

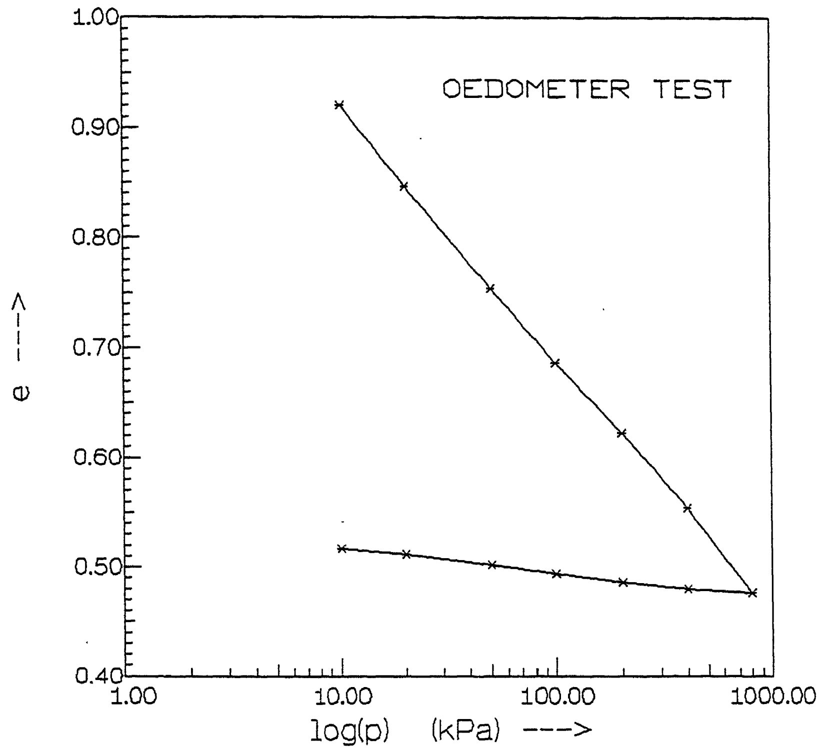


Figure 3.2: Void-ratio – log effective stress curve.

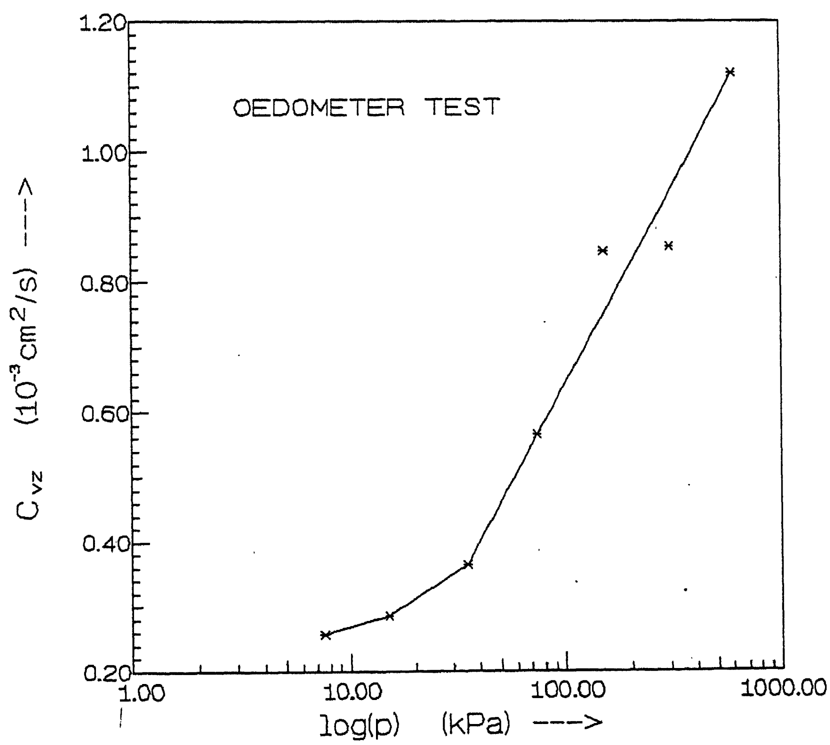


Figure 3.3: Coefficient of consolidation versus effective stress relationship.

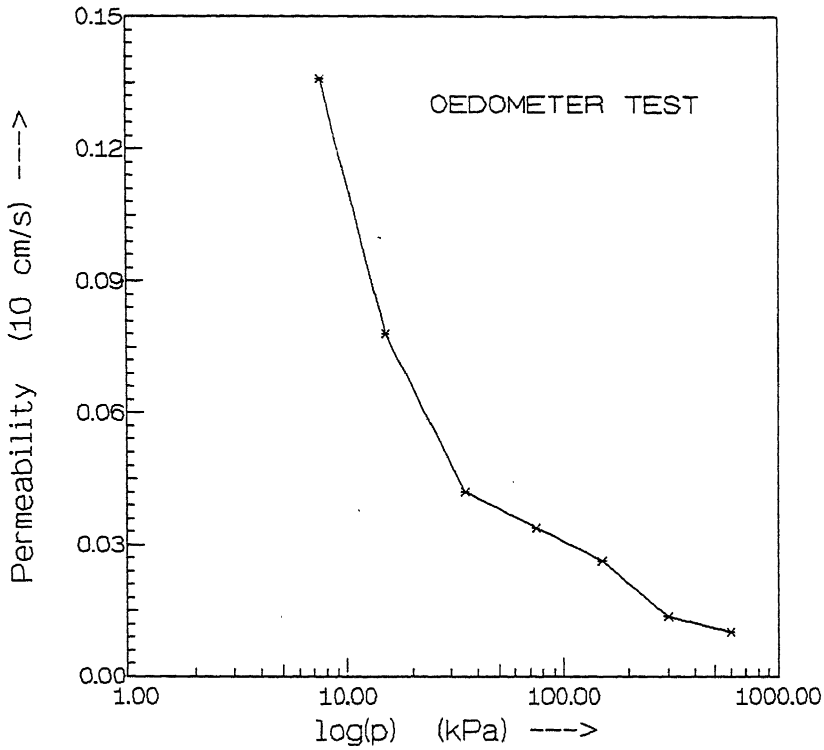


Figure 3.4: Permeability versus effective stress relationship.

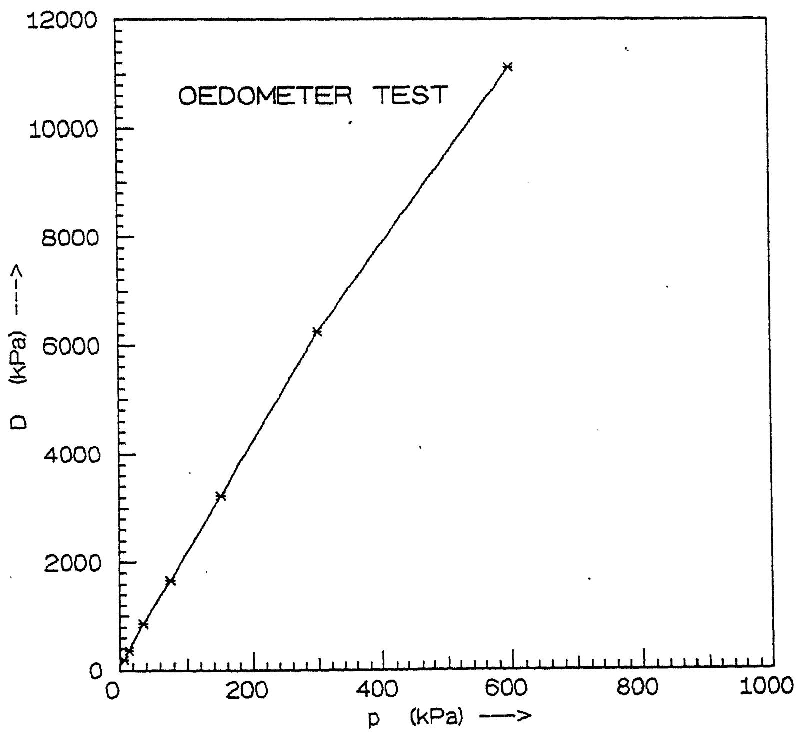


Figure 3.5: Constraint modulus versus effective stress relationship.

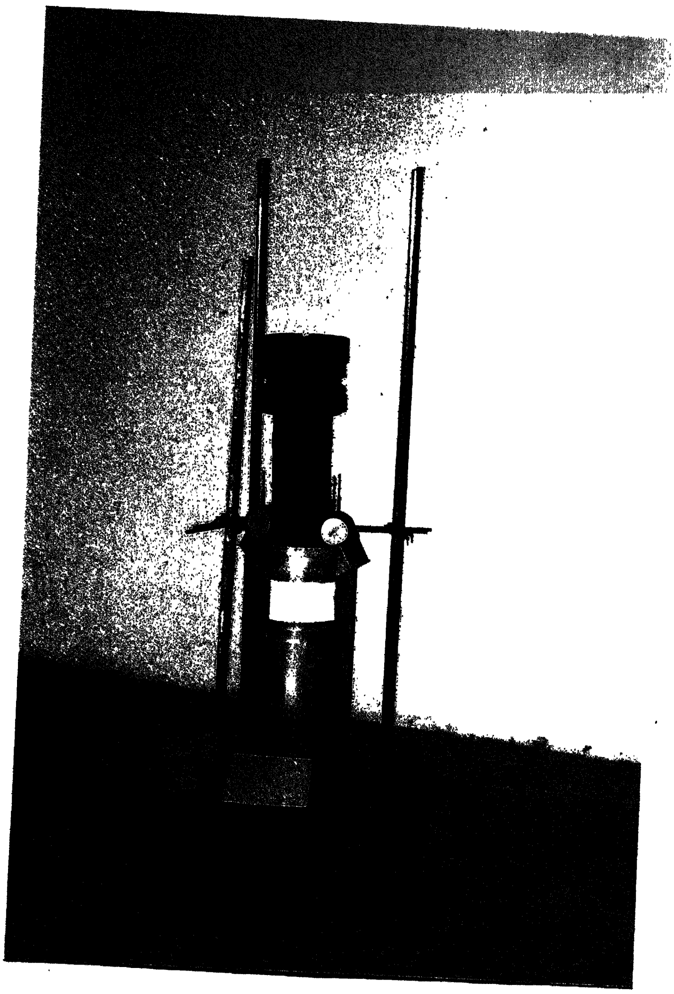


Figure 3.6: View of experimental set-up.

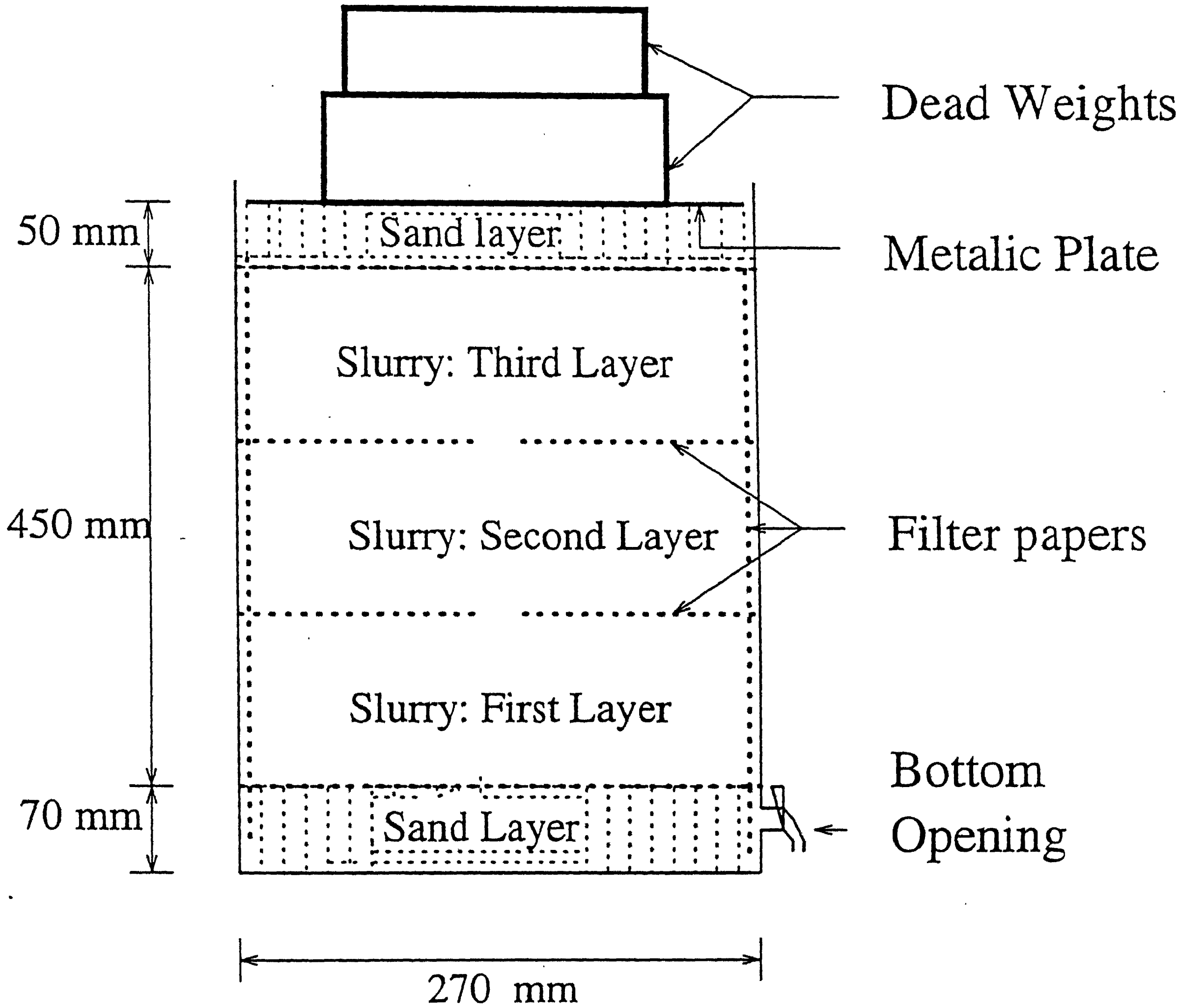


Figure 3.7: Schematic diagram of consolidation system.

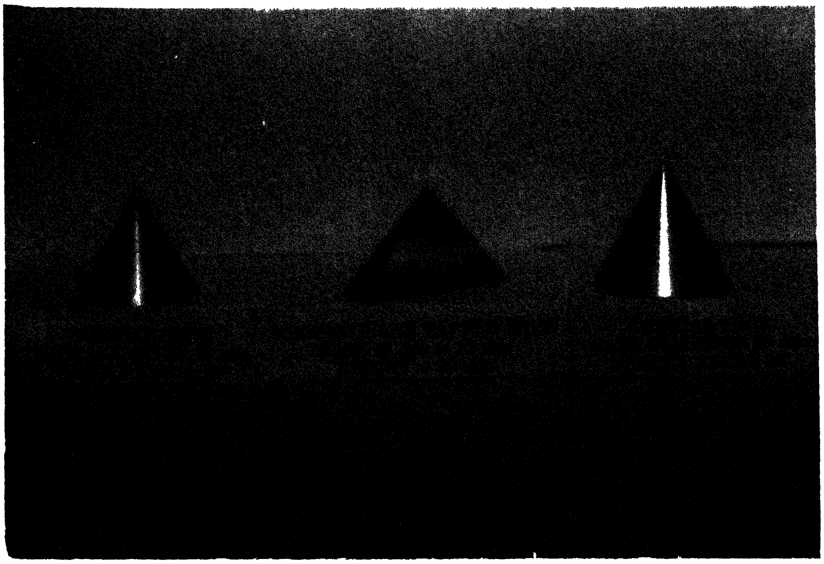


Figure 3.8: View of different types of shoes used in drain installations.

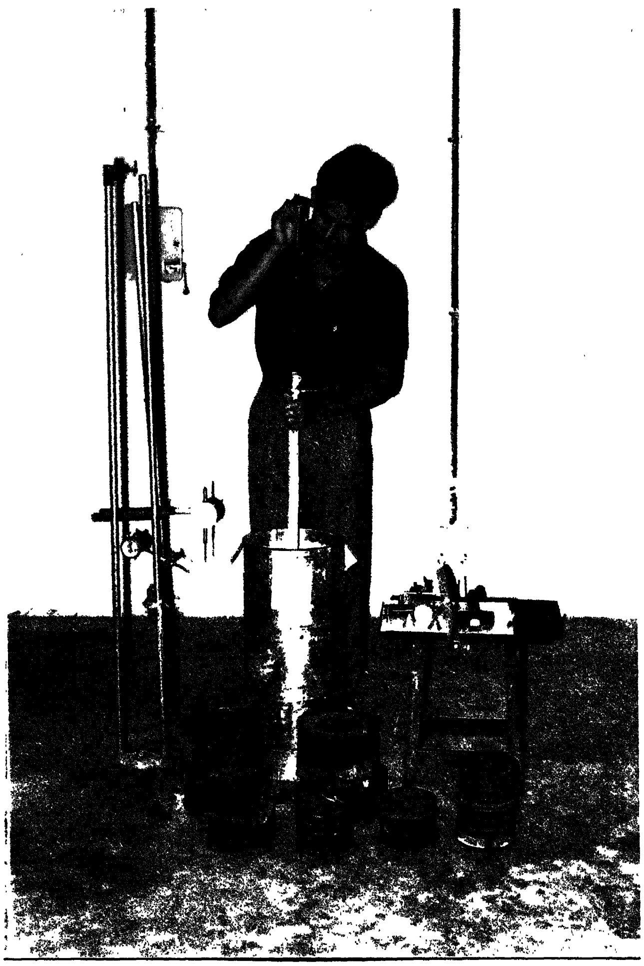


Figure 3.9: Drain installation procedure.

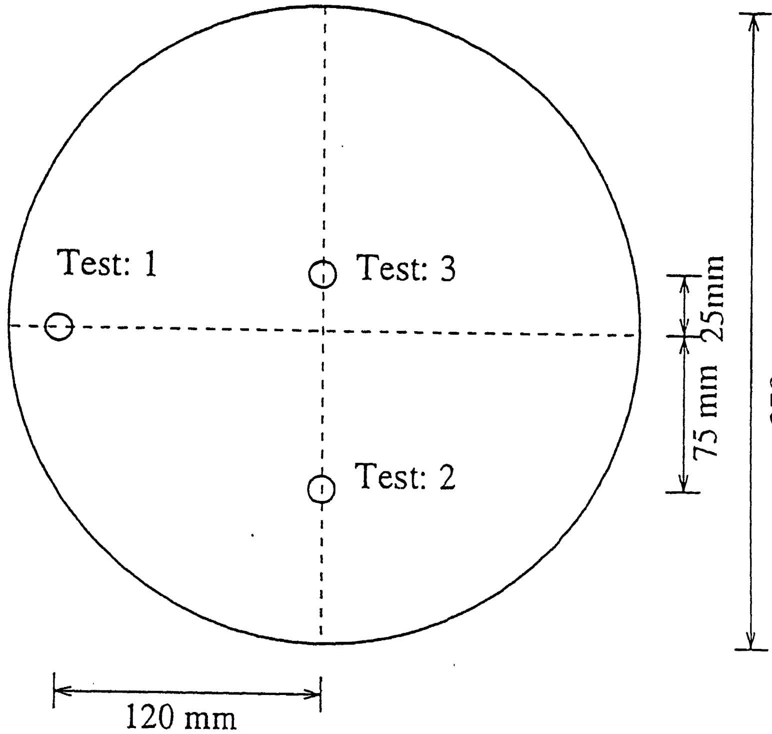


Figure 3.10: Schematic diagram: sequence of cone penetration tests.

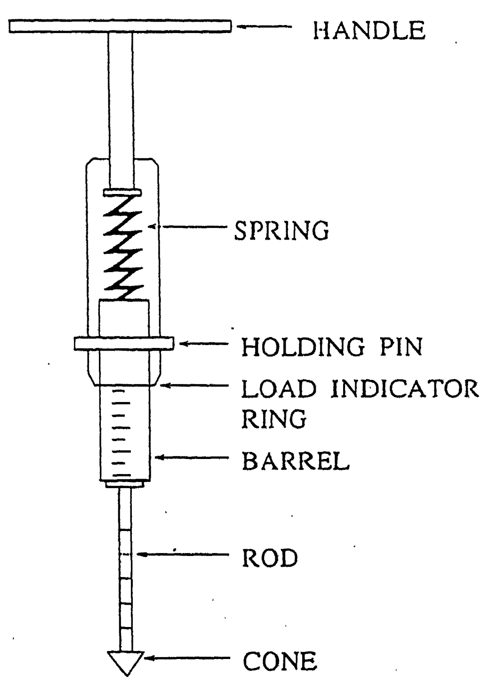


Figure 3.11: Miniature cone penetrometer.

Chapter 4

DATA INTERPRETATION AND DISCUSSION

4.1 General

In this chapter the results obtained from model test, consolidation, reconsolidation and miniature cone penetration tests and water content measurements, are presented, analysed and discussed.

4.2 Comparison Of Model and Oedometer Test Results

In one test, after preparing a sample from slurry (section 3.2.2), continuous load test was carried out without drain. This was done to compare the final settlements and C_v values from oedometer and the model tests. Final settlement estimated based on m_v values from the oedometer test and observed final settlement from the model test, without drain (untreated soil), are presented in Table 4.1. After preparation of slurry, loading was done upto 26.0 kPa in stages: 1.625 kPa, 3.25 kPa, 6.50 kPa, 13.0 kPa and 26.0 kPa. At mid depth of the sample i.e, 22.5 cm from top, the initial stress (p_0) is 4.410 kPa. Stresses at the mid depth have been considered to estimate final settlement based on m_v values from the oedometer test. Observed settlements are on an average 84.3% of the estimated settlements based on Oedometer test. This is may be due to the considerable side friction in the case of the model test. Lubricating oil was applied on the inner surface of the tank to reduce side friction.In oedometer test the height to diameter ratio of the sample is 0.333, while it is 1.667 in the case of model test. Hence side friction may be significant.

From the experimental data of model test on untreated sample, times required

for 90% degree of consolidation (t_{90}) have been calculated using *Taylor's Method* for different load increments. The coefficient of consolidation in the vertical direction (C_{vz}) has been calculated for each load increment. C_{vz} values have also been calculated using Asaoka (1978) method from the time-settlement data. Table-4.2 compares the C_{vz} values estimated from oedometer test with the corresponding C_{vz} values estimated by Taylor's and Asaoka's methods. Fig.4.1 represents C_{vz} versus $\log(p)$ relation for: (i) oedometer test and (ii) model test using untreated sample. The C_{vz} values estimated from the model test data are approximately 95.5% of the corresponding estimated values from the oedometer test. The C_{vz} values estimated from the model test data are slightly smaller than the corresponding estimated values from the oedometer test possibly because of significant side friction in the case of the model test, as dicussed earlier. It can be observed from Table 4.1 that C_{vz} values calculated using Asaoka's method are nearly equal to the corresponding C_{vz} values calculated from Taylor's method. Asaoka's method gives a good estimation of C_{vz} from observed time-settlement data. Final water contents after consolidation test were determined at different depths and radial distances from the center of the drain. The average final water contents were calculated for each stress increment and changes in void ratio were determined. Fig.4.2 represents void ratio (e) versus $\log(p)$ relation for: (i) oedometer test and (ii) model test using untreated sample. Compression index (C_c) obtained from model test is 0.224 which is slightly less than that from Oedometer test, 0.233, because of side friction generated in the model tests, inspite of applying lubricating oil on the inner surface of the tank. In the model, side friction is considerable as the height to diameter ratio of the sample is 1.667, which is five times larger than the corresponding value of oedometer.

4.3 Effects Of Disturbance On Settlement

Soil structure near the drain gets altered and pore pressure is induced due to the disturbance caused by vertical drain installation. The disturbance effects cause the in-situ soil to undergo larger settlements of the ground compared to the untreated ground (Terzaghi and Peck, 1967; Madhav et al., 1995). Model tests have been carried out to study the disturbance effects, on the drain treated ground, caused by vertical drain installation (section 3.3).

Experimental time-settlement curves for the cases viz. OCR equal to 1.75 and 2.5 and OCR equal to 2.5 with aging for the load increments: (i) 0.0 kPa to 1.625 kPa (reloading), (ii) 6.5 kPa to 13.0 kPa (vergin loading) and (iii) 13.0 kPa to 26.0 kPa (vergin loading) are presented in Fig.s 4.3, 4.4 and 4.5 respectively. Tables 4.3, 4.4 and 4.5 show the ultimate settlements. From Tables 4.3, 4.4 and 4.5 it is observed that in some cases ultimate settlement values obtained for *test on untreated sample* are more than the corresponding ultimate settlement values for *test on drain treated sample*. In the model tests silty clay has been used. During drain installation soil may be partly densified. In the field vertical drains are often installed in saturated clay.

So, the possibility of soil getting densified in the field due to vertical drain installation is very less. In the model tests two phenomenon: (i) densification and (ii) disturbance occur during drain installation, while in the field mainly disturbance occurs. Due to densification during drain installation less settlement may have occured in those cases of model tests.

A comparison of ultimate settlements for virgin loadings is presented in Table 4.6. Settlement ratios have been calculated and compared to study the effects of shoe size, OCR and aging on disturbance due to drain installation. Settlement ratio is defined as the ratio of ultimate settlement of the drain treated ground to the corresponding ultimate settlement of the untreated ground. Increasing settlement ratio means increasing disturbance due to drain installation. From Table 4.6 it can be observed that for stress increment *13.0 kPa to 26.0 kPa* for the cases: (i) OCR equal to 2.5 and (ii) OCR equal to 2.5 with aging, ultimate settlement of drain treated ground is larger than the ultimate settlement of untreated ground — for all types of shoes. This fact clearly indicates that disturbances due to drain installation increase ultimate settlement of the drain treated ground, as reported by Madhav et al. (1995). From Table 4.6 it can also be observed: (i) with the increase in OCR and (ii) aging, disturbances due to drain installation increases for the same shape and size of shoe. In each of the Tables 4.3, 4.4, 4.5 and 4.6, it can be observed that ultimate settlement increases with the increasing shoe size as reported by Bergado et al. (1993).

4.4 Cone Penetration Tests & Water Content Measurements

To study effects of drain installation on consolidation of drain treated ground, miniature cone penetration tests were carried out after consolidation and water contents (w_c) were measured at different depths and radial distances from the center of the drain. In this section results of miniature cone penetration tests and w_c measurements are discussed.

4.4.1 Cone Penetration Test Results

Miniature cone penetration tests were done in each model after final unloading. Cone penetration tests were done at different depths and at different radial distances as discussed in Section 3.2.5. Cone penetration resistance (q_c) versus depth curve is plotted for all the cases studied in the model tests. Cone penetration resistance versus depth plots for the cases:

1. Fig.4.6(a): Untreated sample and OCR equal to 2.5.
2. Fig.4.7(a): With drain, shoe of 30mm base diameter and OCR equal to 1.75.

3. Fig.4.8(a): With drain, shoe of 30mm base diameter and OCR equal to 2.5.
4. Fig.4.9(a): With drain, rectangular tappered shoe, OCR equal to 2.5 and aging.
5. Fig.4.10(a): With drain, shoe of 40mm base diameter and OCR equal to 2.5.
6. Fig.4.11(a): With drain, shoe of 40mm base diameter OCR equal to 2.5 and aging.

are presented. It can be observed that q_c increases with depth, due to depth effect. With the increase in depth effective stress increases and q_c increases. This is termed as depth effect. It is also observed that q_c values obtained at the depths of intermediate filter papers are larger than those in the adjacent depths. Filter papers were placed at regular intervals in the soil to accelarate consolidation. Near the filter papers local consolidation has occured along with radial consolidation (for with drain cases). It is similar to consolidation of varved clay (Rowe, 1964). Due to this local consolidation, near the filter papers degree of consolidation achieved is more and q_c values obtained at these locations are larger than those at the adjacent depths.

Tests On Untreated Sample

In the model tests on untreated sample, there should not be any variation in the degree of consolidation with radial distance from center. The q_c values for without drain cases vary slightly with the radial distance from center, Fig.4.6(a). This may be due to interference effect of previous cone penetration test(s) on the next cone penetration test(s) and natural variations.

Tests With Drains

In general, for the model tests with drains, q_c values obtained at a particular depth and radial distance are larger than the corresponding q_c values for the model test without drain. From Fig.s 4.7(a), 4.8(a), 4.9(a), 4.10(a) and 4.11(a) it can be observed that at a given depth (except near the filter papers) q_c values decrease with increasing radial distance. Near the vertical drain, degree of consolidation achived is more compared to the degree of consolidation of the far away zones. So, q_c values obtained near the drain are more compared to those at far away zones.

4.4.2 Water Content Profile

After cone penetration tests, water contents (w_c) were measured at different depths and radial distances from the center (Section 3.2.5). Variation of w_c with depth is plotted and presented for the following cases:

1. Fig.4.6(b): Untreated sample and OCR equal to 2.5.
2. Fig.4.7(b): With drain, shoe of 30mm base diameter and OCR equal to 1.75.
3. Fig.4.8(b): With drain, shoe of 30mm base diameter and OCR equal to 2.5.
4. Fig.4.9(b): With drain, rectangular tappered shoe, OCR equal to 2.5 and aging.
5. Fig.4.10(b): With drain, shoe of 40mm base diameter and OCR equal to 2.5.
6. Fig.4.11(b): With drain, shoe of 40mm base diameter OCR equal to 2.5 and aging.

Due to the depth effect, as discussed earlier w_c decreases with depth. From w_c versus depth plots it is observed that near the filter papers w_c values are smaller than that of at the neghibouring depths. This due is similar to the varved clay phenomenon as discussed earlier in Section 4.3.1.

Tests On Untreated Sample

Fig.4.6(b) shows a set of w_c versus depth relationships for model test on untreated sample with OCR equal to 2.5. In the model tests on untreated samples, variation of w_c with radial distance is negligible, as it should be.

Tests With Drains

Comparing Fig.4.6(b) with Fig.4.8(b), it can be observed that for the model test with drain, OCR equal to 2.5 and drain installed with conical shoe of 30 mm base diameter, w_c values obtained at a particular depth and radial distance are smaller than the corresponding values for the model test without drain, i.e, model test on untreated sample. Similar test results were obtained for other cases also. From Fig.s 4.7(b), 4.8(b), 4.9(b), 4.10(b) and 4.11(b) it is observed that at a particular depth (except near filter papers), w_c value increases with increasing radial distance from the center of the drain. Near the vertical drain degree of consolidation achived is more compared to the far away zones.

The initial w_c varies from 42.9% to 43.3%. The percentage reduction in initial w_c is calculated for each case. The results are summerised in Table-4.8. From Table-4.8, it can be observed that percentage reduction in w_c due to consolidation increases with:

- increase in OCR,
- aging,
- drain installation and

Table 4.1: Comparison of estimated and observed final settlements.

Stress Increment At Mid Depth (kPa)	Final Settlement (cm)		$100(S_{f,m}/S_{f.o})$
	Estimated From Oedometer Test $S_{f.o}$	Observed (Untreated Sample) & OCR = 1), $S_{f,m}$	
4.410 to 6.035	0.928	0.778	83.8
6.035 to 7.660	0.639	0.535	83.7
7.660 to 10.910	0.876	0.733	83.1
10.910 to 17.410	1.084	0.910	83.9
17.410 to 40.410	1.267	1.100	86.8

- increase in size of shoe used in drain installation, i.e, increasing disturbance due to drain installation.

4.4.3 Correlation Between Water Content & Cone Penetration Resistance

Water contents (w_c) and cone penetration resistances (q_c) were determined at different depths and radial distances from the center, as discussed earlier. The correlation between w_c and q_c is studied by plotting the (w_c, q_c) values at corresponding locations (Fig.4.12). The plot shows some scatter but a best fit line can be drawn through the plotted points. Eq.4.1 represents the correlation between w_c and q_c:

$$q_c = 364.8 - 9.1w_c \tag{4.1}$$

where q_c is in kPa and w_c is expressed in percentage. Eq.4.1 can be used to determine q_c values from w_c values or vice versa. Change in OCR and natural variation in soil may have affected w_c versus q_c relationship. The relationship is obtained for Kanpur silty clay. Hence, it may be used only for the same soil.

On the basis of time-settlement data, cone penetration tests and water content measurements and analysis of the test results it may be concluded that size of shoe, OCR and aging affect disturbance due to drain installation and hence settlement of the drain treated ground. Disturbance due to drain installation and cosequently settlement of the drain treated ground increases with the increasing size of shoe, OCR and aging.

Table 4.2: Comparison of C_{vz} values.

Stress Level At Mid Depth (kPa)	Coefficient Of Consolidation (cm^2/s)		
	Oedometer Test (Estimated)	Model Test (Untreated Sample)	
		Square Root Plot	Asaoka Method
5.223	2.520×10^{-4}	2.445×10^{-4}	2.492×10^{-4}
6.848	2.570×10^{-4}	2.534×10^{-4}	2.603×10^{-4}
9.285	2.620×10^{-4}	2.555×10^{-4}	2.493×10^{-4}
14.160	2.800×10^{-4}	2.628×10^{-4}	2.685×10^{-4}
23.910	3.180×10^{-4}	2.997×10^{-4}	3.012×10^{-4}

Table 4.3: Settlements for OCR equal to 1.75.

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm Dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm Dia. Shoe, Base Area 12.566 cm^2
0.0	1.625*	0.086	0.387	0.490	0.497
1.625	3.25*	0.151	0.667	0.571	0.560
3.25	6.5	0.668	0.560	0.626	0.690
6.5	13.0	0.849	0.796	0.842	0.818
13.0	26.0	1.086	0.969	1.015	1.077

* Reloadling

Table 4.4: Settlements for OCR equal to 2.5

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm Dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm Dia. Shoe, Base Area 12.566 cm^2
0.0	1.625*	0.088	0.563	0.603	0.647
1.625	3.25*	0.060	0.474	0.452	0.582
3.25	6.5*	0.165	0.625	0.676	0.647
6.5	13.0	0.848	0.776	0.819	0.863
13.0	26.0	0.942	0.992	0.991	1.122

* Reloadling

Table 4.5: Settlements for OCR equal to 2.5 and aging.

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm Dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm Dia. Shoe, Base Area 12.566 cm^2
0.0	1.625*	0.070	0.550	0.620	0.660
1.625	3.25*	0.050	0.480	0.525	0.580
3.25	6.5*	0.151	0.610	0.630	0.655
6.5	13.0	0.818	0.785	0.832	0.872
13.0	26.0	0.902	0.980	1.010	1.132

* Reloadling

Table 4.6: Comparison of ultimate settlements.

OCR, Aging	Stress Increment (kPa) (Vergin Loading)	Settlement Ratio			
		Untreated Sample	30 mm Dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm Dia. Shoe, Base Area 12.566 cm^2
1.75	6.5 to 13.0	1.0	0.938	0.992	0.963
	13.0 to 26.0	1.0	0.892	0.934	0.992
2.5	6.5 to 13.0	1.0	0.915	0.966	1.018
	13.0 to 26.0	1.0	1.053	1.052	1.191
2.5 & Aging	6.5 to 13.0	1.0	0.960	1.017	1.066
	13.0 to 26.0	1.0	1.086	1.120	1.253

Table 4.7: Comparison of water contents.

Shoe Type	OCR = 1.75			OCR = 2.5			OCR = 2.5 & Aging		
	A	B	C	A	B	C	A	B	C
*	42.2	31.8	26.38	43.3	31.1	28.18	43.1	30.3	29.70
1	43.2	30.9	28.47	43.1	29.7	31.09	43.2	29.7	31.25
2	43.0	30.5	29.07	43.0	29.6	31.16	42.9	29.4	31.40
3	43.3	30.3	29.53	42.9	29.3	31.70	42.9	28.8	32.87

* Without drain.

1: Conical shoe of 30 mm base diameter.

2: Rectangular shoe of base size 48mm X 15mm.

3: Conical shoe of 40 mm base diameter.

A: Initial average water content

B: Final average water content

C: Perecentage reduction in water content

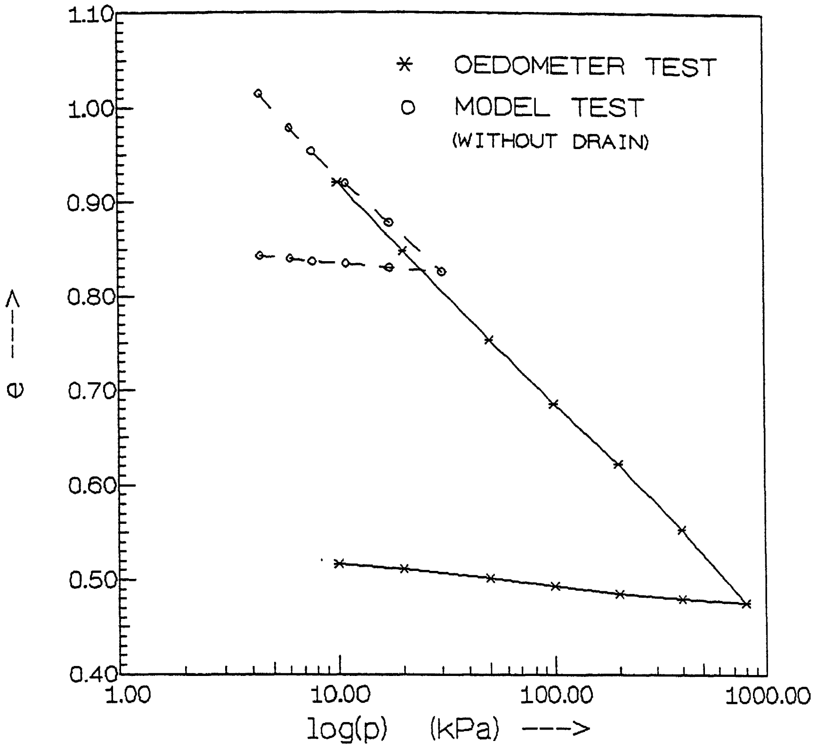


Figure 4.1: Void-ratio versus log effective stress relationship.

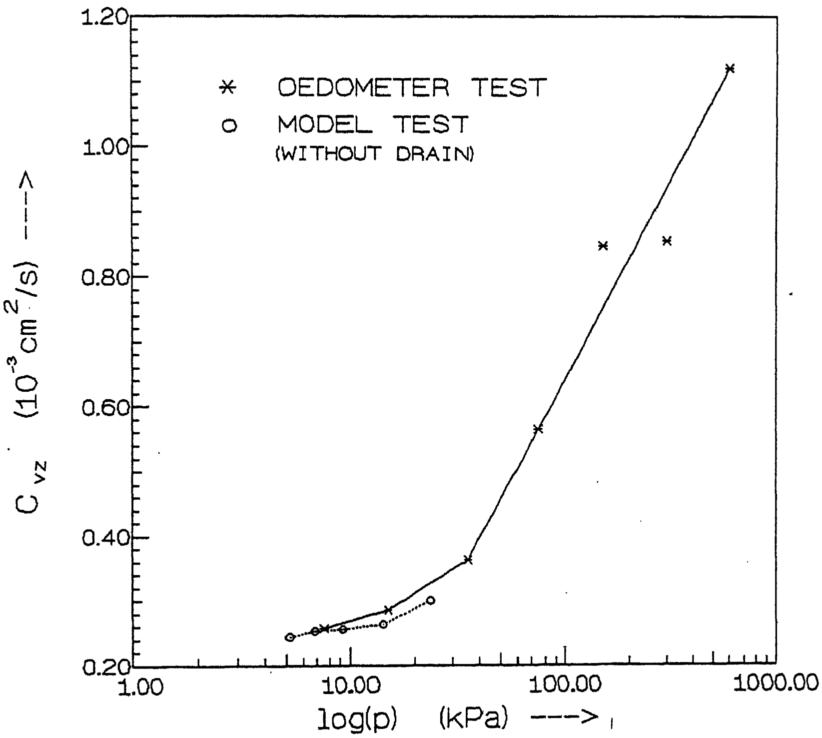


Figure 4.2: Coefficient of consolidation versus effective stress relationship.

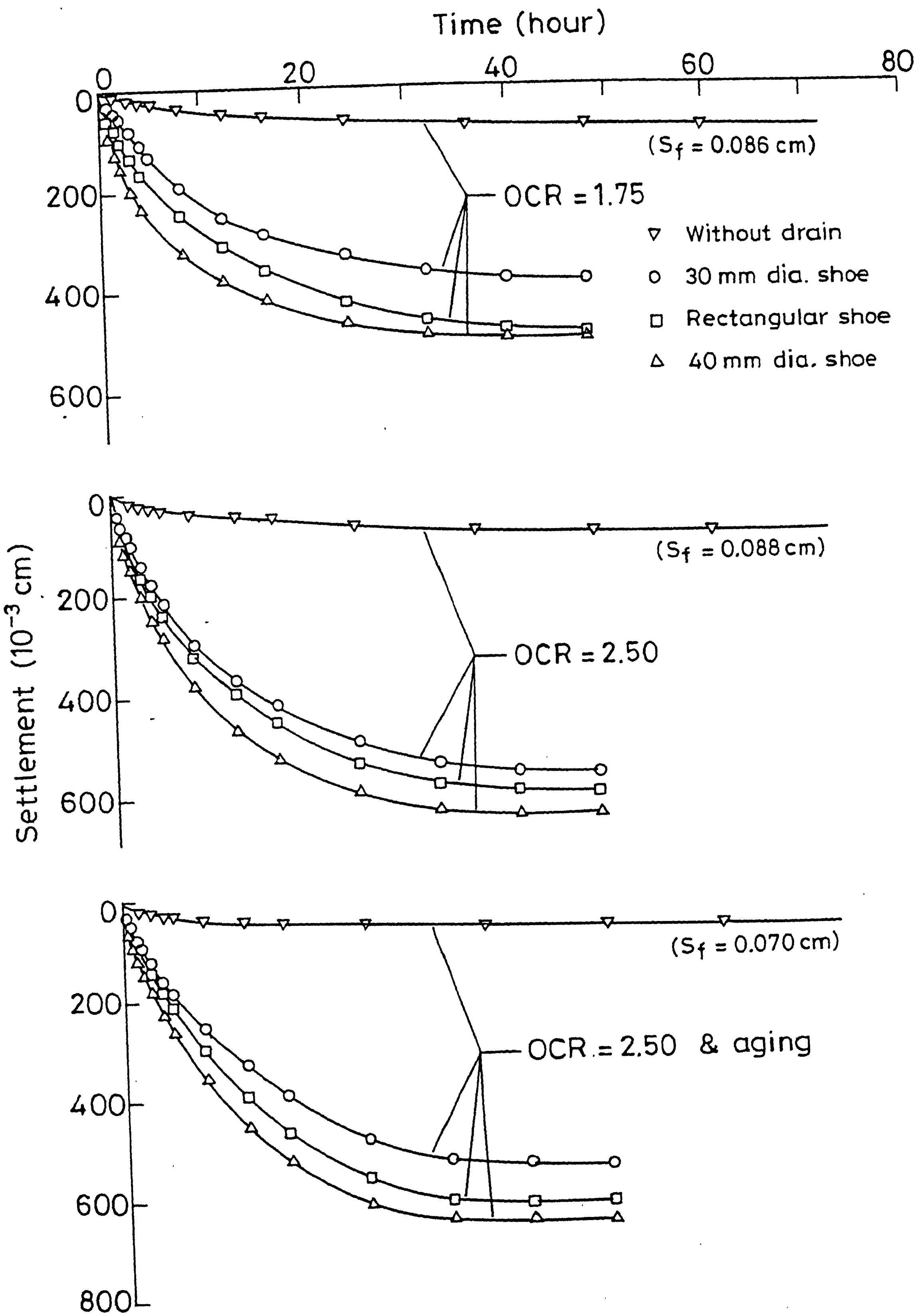


Figure 4.3: Time-settlement relationships for stress increment 0.0 kPa to 1.625 kPa (reloading).

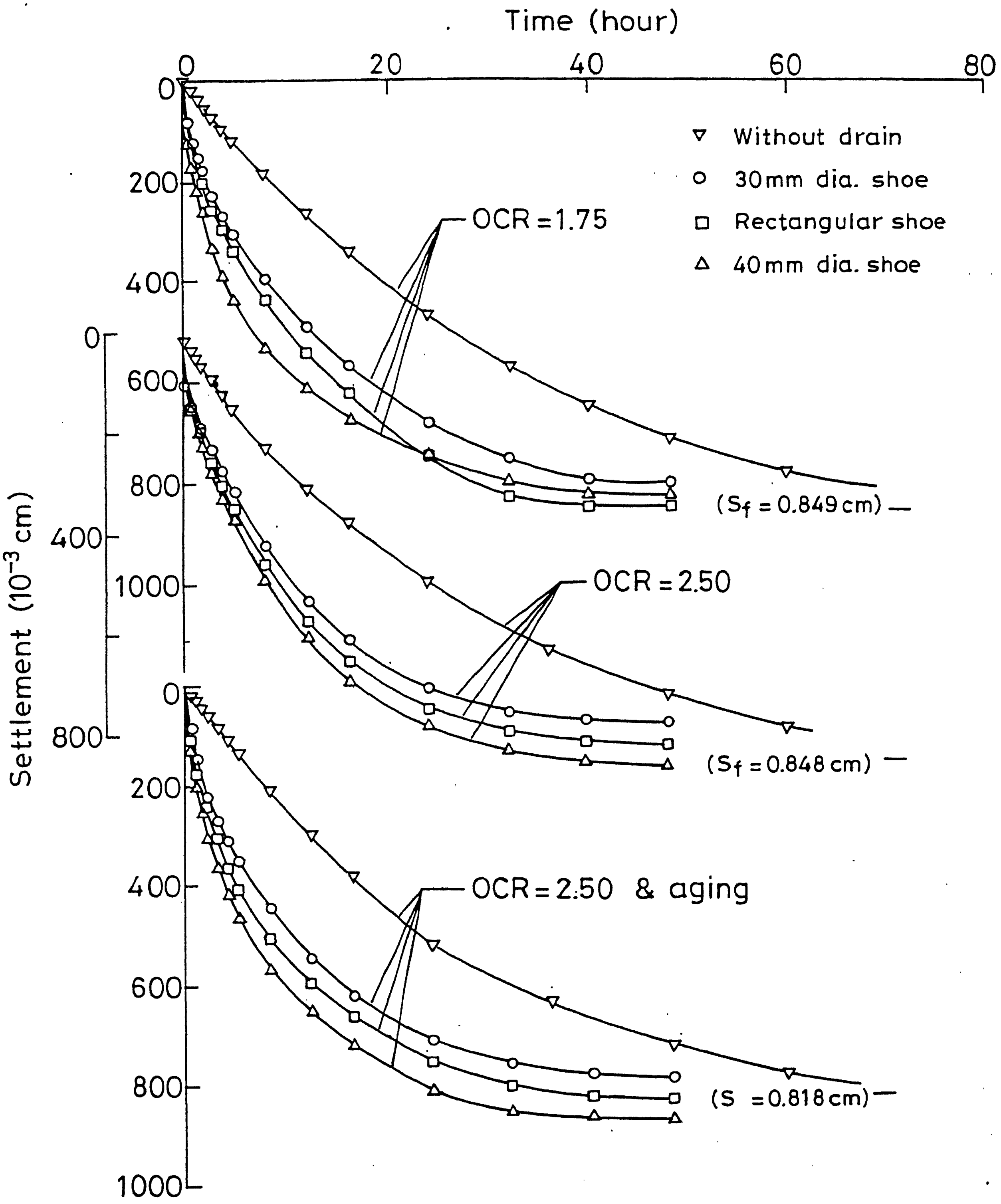


Figure 4.4: Time-settlement relationships for stress increment 6.5 kPa to 13 kPa (virgin loading).

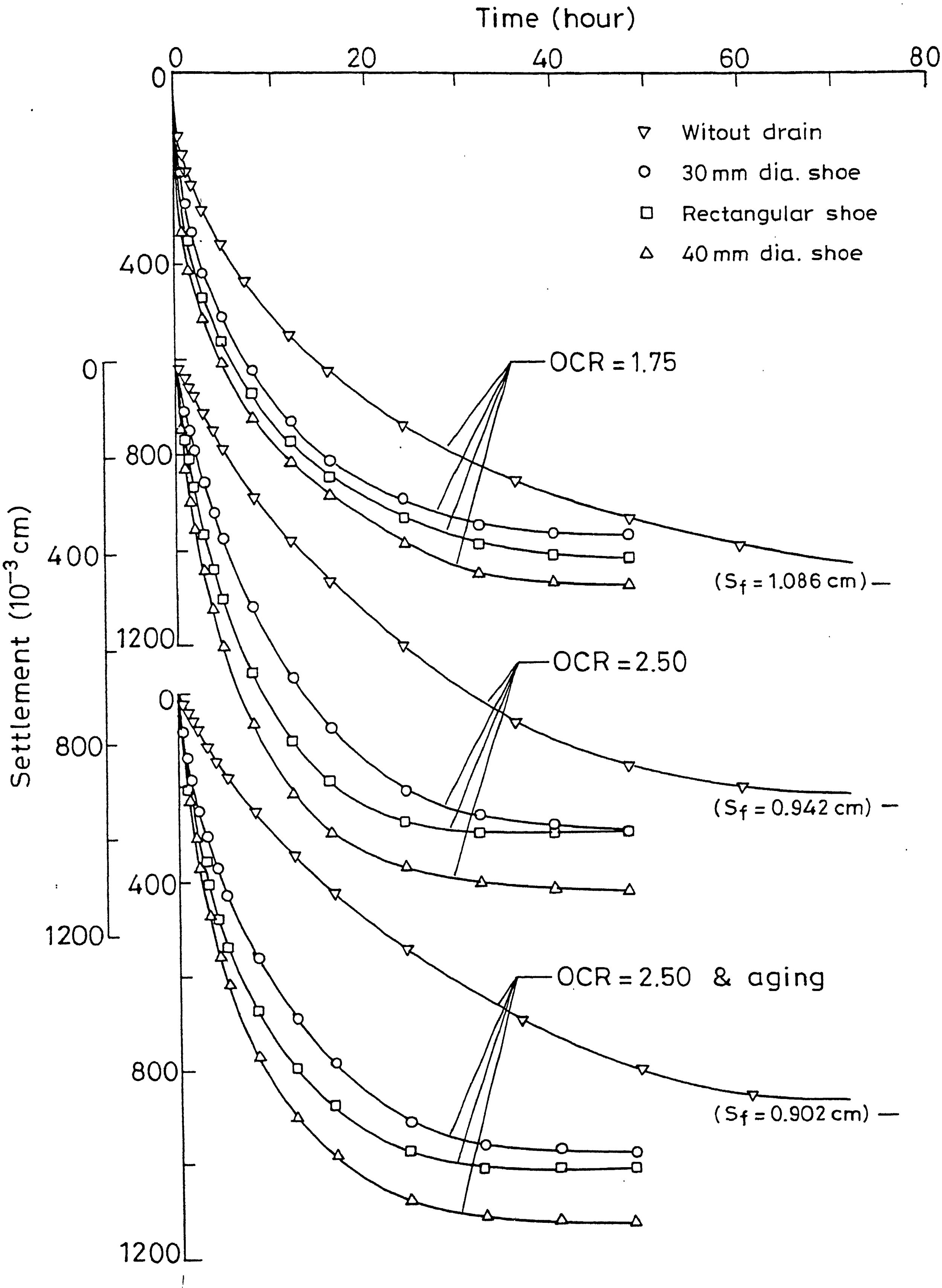


Figure 4.5: Time-settlement relationships for stress increment 13 kPa to 26 kPa (virgin loading).

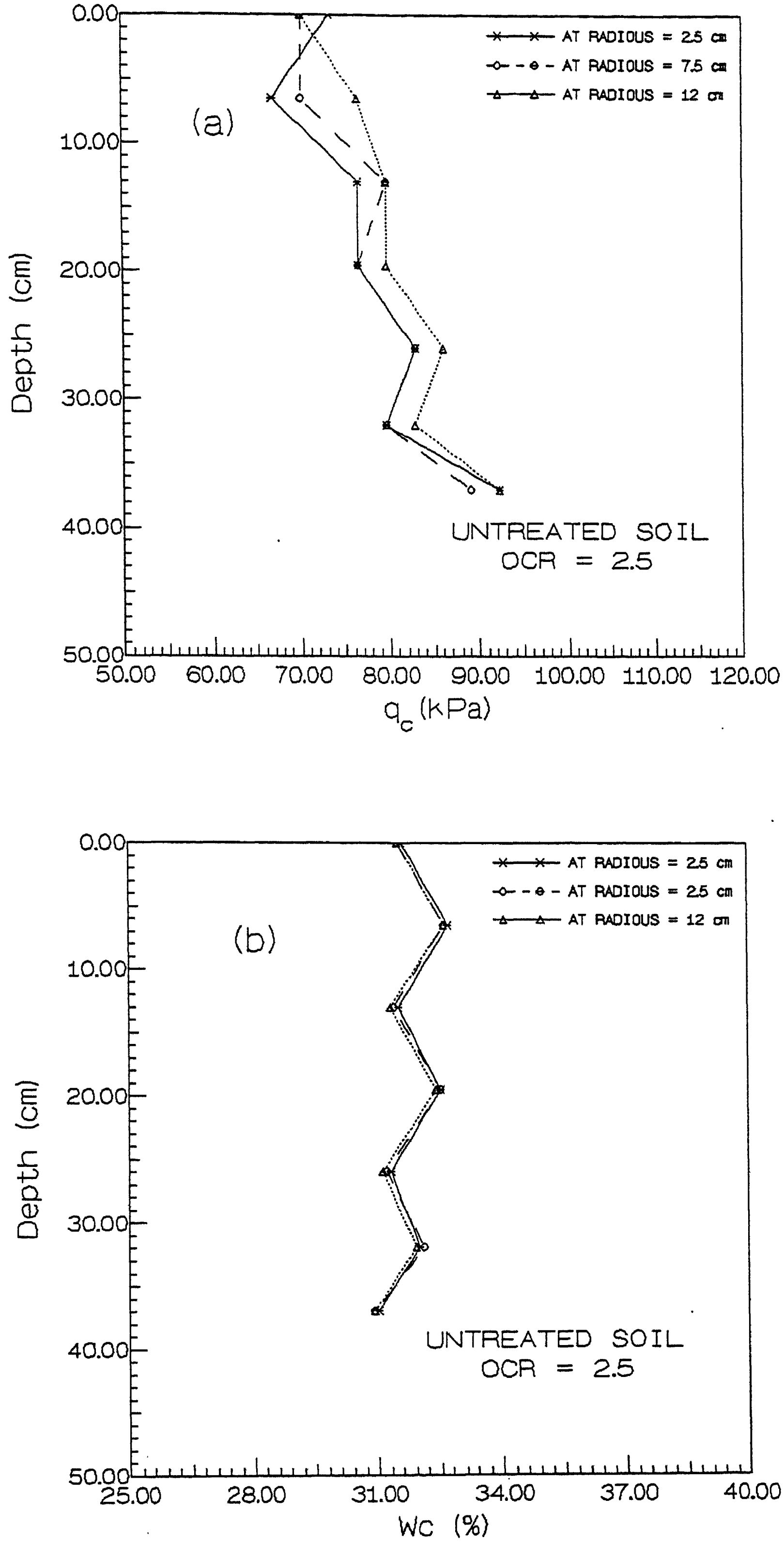


Figure 4.6: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.

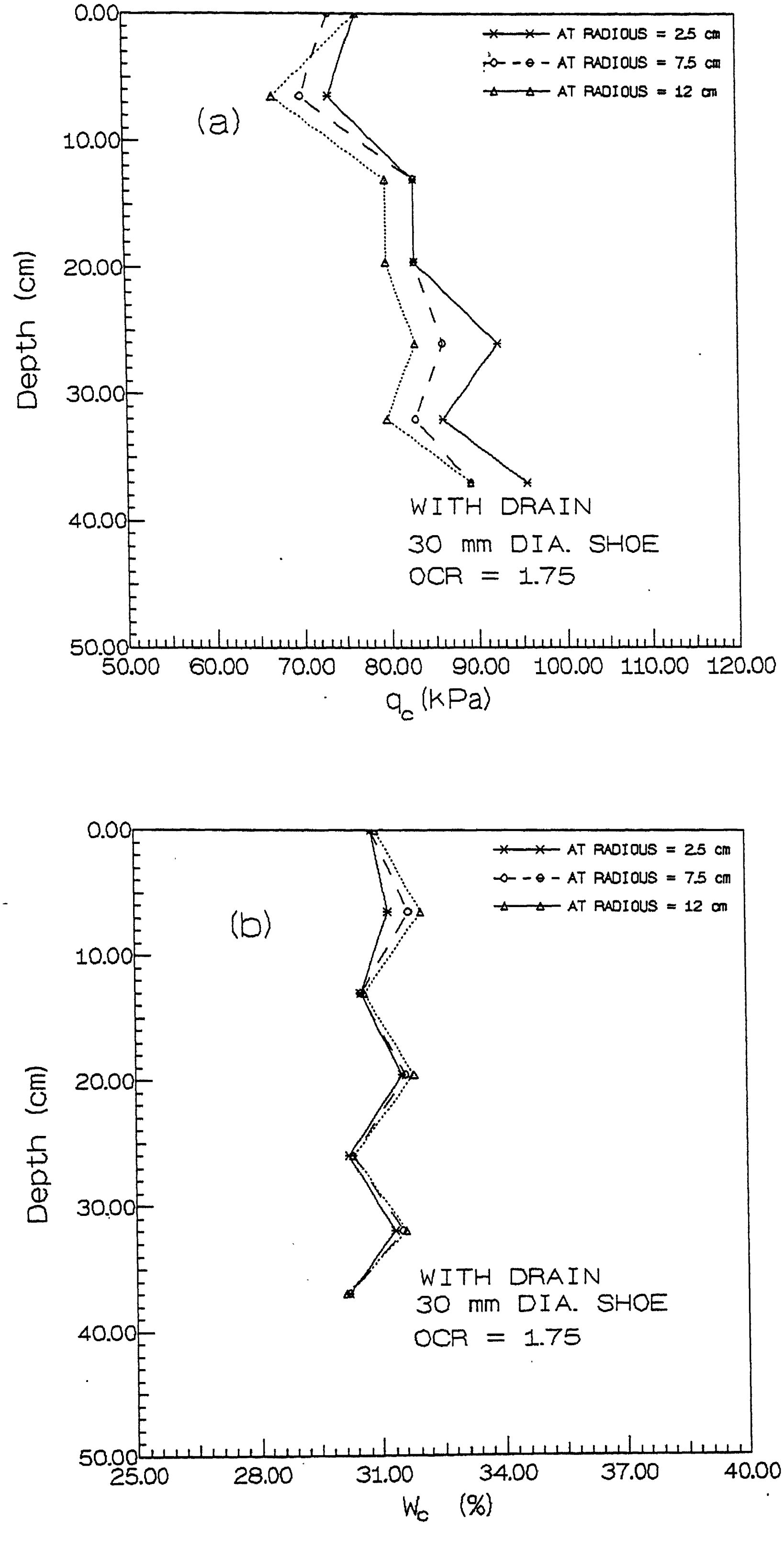
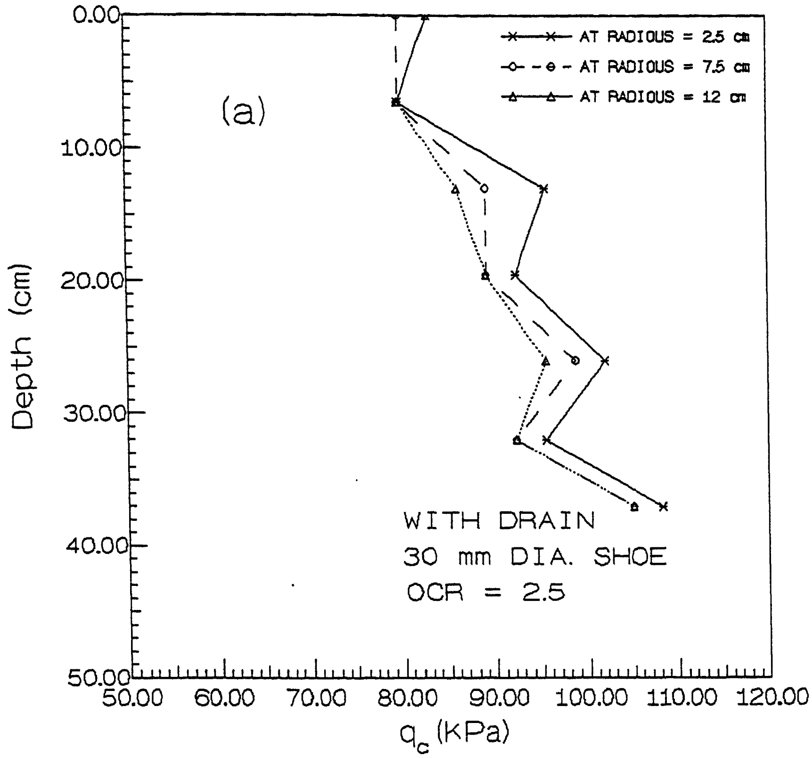


Figure 4.7: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.



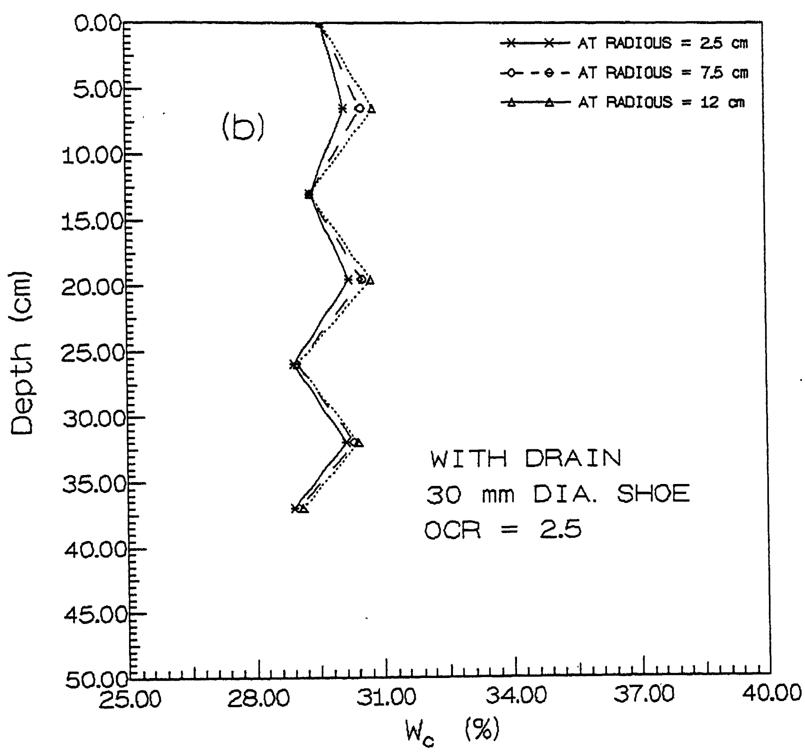
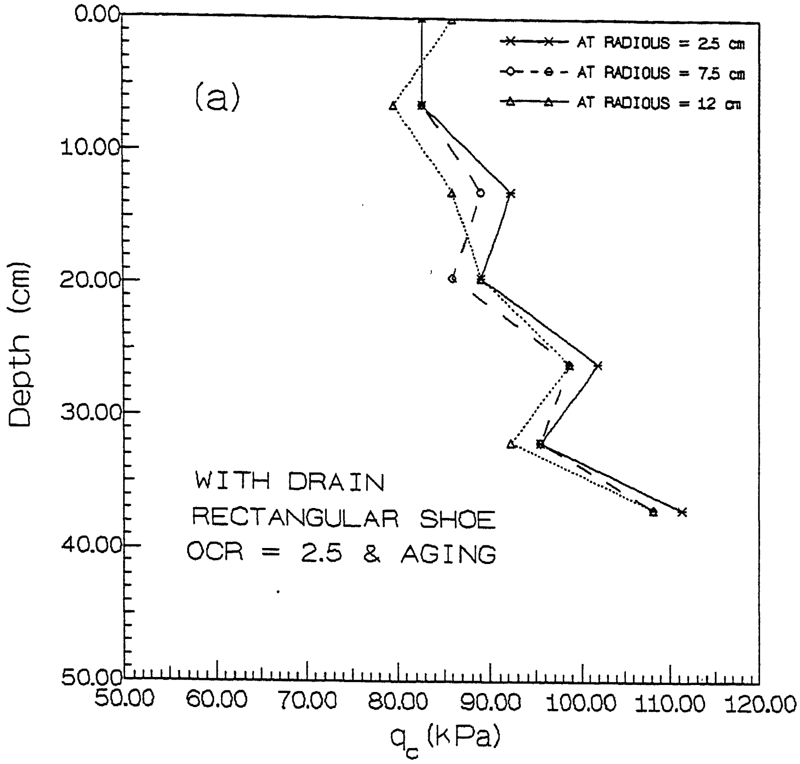


Figure 4.8: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.



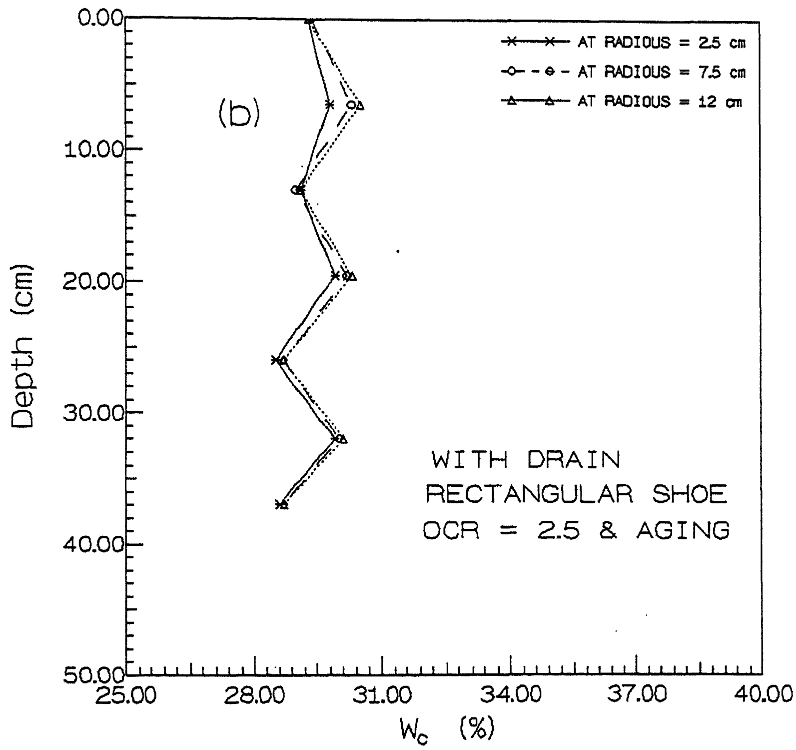


Figure 4.9: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.

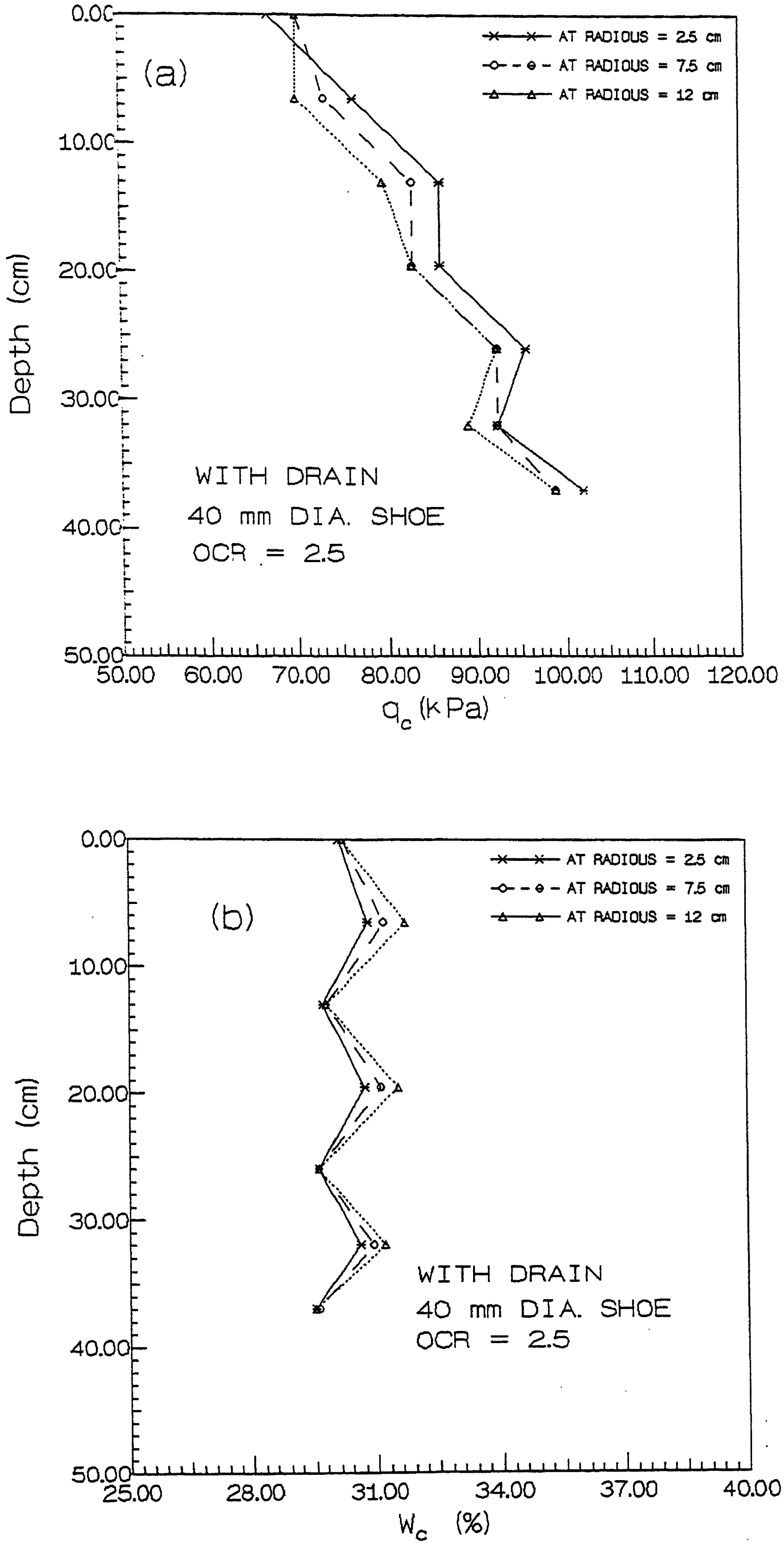
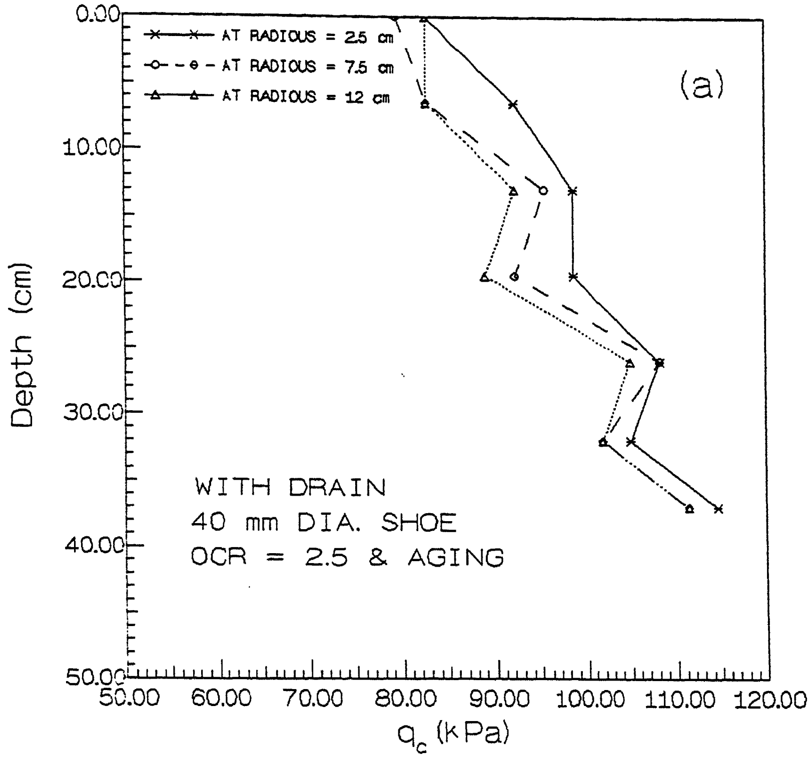


Figure 4.10: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.



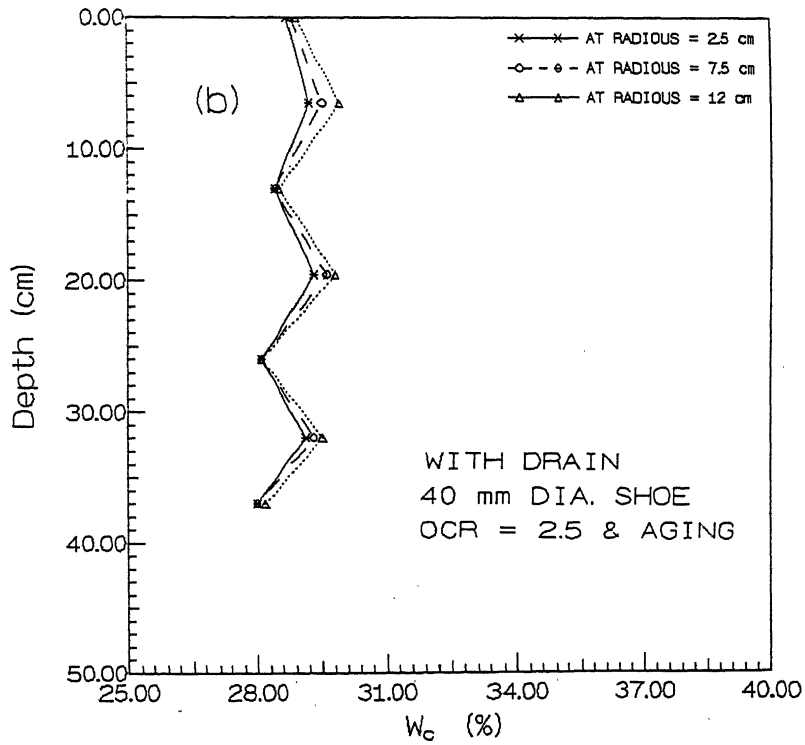
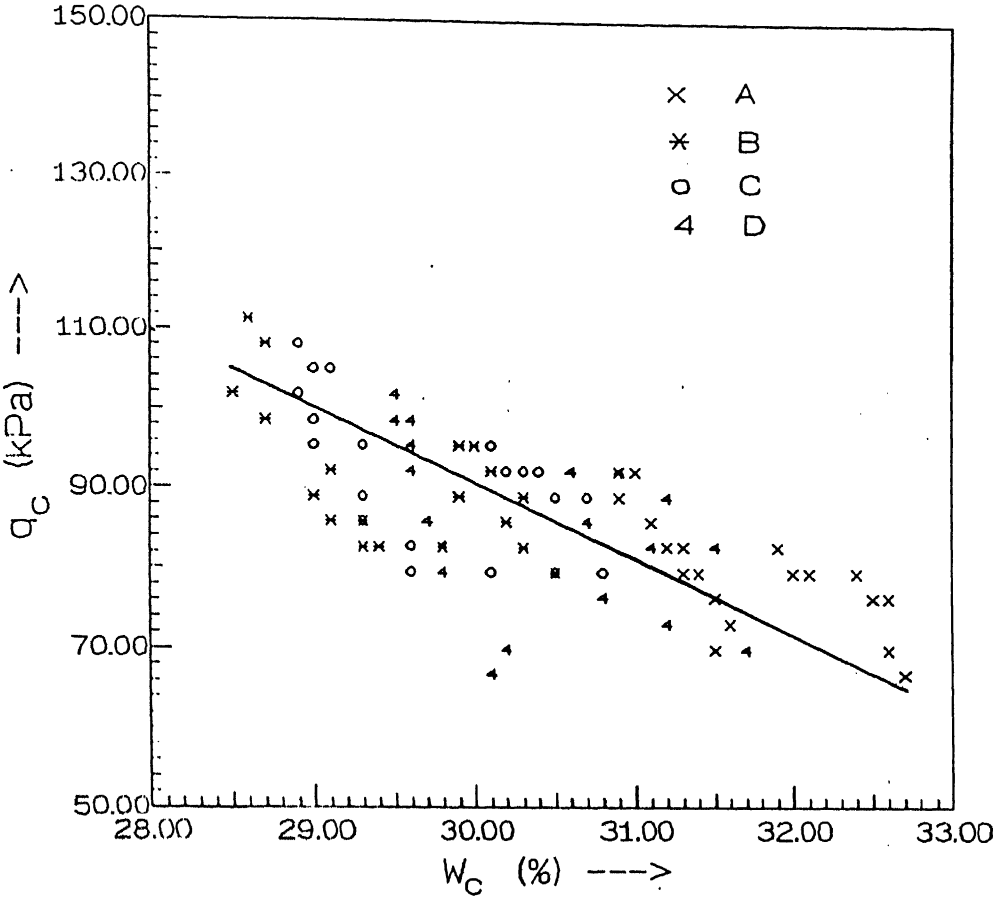


Figure 4.11: Variation of: (a) cone penetration resistance, q_c and (b) water content, w_c with depth.



A: Untreated sample & OCR = 2.5
B: With drain, 30 mm dia. shoe & OCR = 2.5
C: With drain, Rectangular shoe & OCR = 2.5 and aging
D: With drain, 40 mm dia. shoe & OCR = 1.75

Figure 4.12: Water content versus cone penetration resistance relationship.

Chapter 5

ESTIMATION OF LONG TERM SETTLEMENT FROM IN-SITU MEASUREMENTS

5.1 General

To solve the consolidation equation theoretically we need to determine the relevant soil parameters, and assume appropriate initial and boundary conditions. In actual field conditions values of soil parameters (coefficient of consolidation C_v, m_v, k, etc.) and initial distribution of pore water pressure may be quite different from the assumed values. Sand lenses may be present within the soft clay strata, which may be unnoticed during soil investigation. For a large site, thickness of the soil strata may vary considerably. Consedering all these aspects observational procedures to estimate consolidation settlement have been developed. Asaoka (1978) and Kodandaramaswamy & Narasimha Rao (1980) present observational methods to estimate ultimate settlement. A new observational method is proposed modifying Asaoka's method and compared with the Asaoka's and Hyperbolic (Kodandaramaswamy & Narasimha Rao, 1980) methods.

5.2 Asaoka Method

Asaoka (1978) has presented two kinds of observational procedures to calculate consolidation settlement: (a) a graphical method and (b) a reliabilistic approach. The graphical method is very simple while for using the reliabilistic approach, help of a computer is needed. The Asaoka Method is based on the fact that one-dimentional consolidation settlement at times 0, Δt, $2\Delta t$, $3\Delta t$, etc. can be expressed mathematically in the general form:

$$s_n = \beta_0 + \sum_{i=0}^{i=w} \beta_n s_{n-i} \tag{5.1}$$

where s_n is the settlement at time t = $n\Delta t = t_n$,
$\beta_n (n = 0,\ 1,\ 2,\ ...n)$ are unknown parameters.

Eq.5.1 gives an idea of observational settlement prediction. For w = 1, a first order approximation is obtained:

$$s_n = \beta_0 + \beta_1 s_{n-1} \tag{5.2}$$

Due to constant external load, settlement values (s_0, s_1, s_2, ... s_n), at constant time intervals are observed. The n points, (s_n, s_{n-1}) for n = 1, 2, 3, ... n are plotted on the (s_n, s_{n-1}) co-ordinate axes. It is visually seen whether all these points lie on a straight line as Eq.5.2 suggests or not. A line is drawn through the points plotted, extrapolated, and intersected with the 45° line. The intersection point represents the ultimate settlement (s_f), Fig.5.1. With a constant surcharge load, a break in the slope of the plot indicates the end of the primary settlement. The continuation of the line represents secondary settlement; the final settlement at the intersection with the 45° line then includes secondary settlement.

Asaoka (1978) has also presented an expression for time-settlement relationship considering the first order approximation of Mikasa's (1963) consolidation equation. The time-settlement relationship is given as:

$$s(t) = s_f - (s_f - s_0)\exp(-t/c) \tag{5.3}$$

where $s(t)$= settlement at any time t,
s_f= ultimate settlement,
s_o= settlement at time t equal to zero,
$c = \frac{2H^2}{3C_v}$, for double drainage,
$= \frac{H^2}{2C_v}$, for single drainage,
C_v= coefficient of consolidation
and H= length of drainage path.

From Eq.5.2, a time settlement relation can be obtained, as given below:

$$s(t) = \frac{a_0}{a_1}(1 - \exp(a_1 t)) \tag{5.4}$$

where $a_0 = \dfrac{\beta_0}{\Delta t}$

$a_1 = \dfrac{(\beta_1 - 1)}{\Delta t}$

and $\Delta t=$ constant time interval between consecutive settlement observations

5.3 Hyperbolic Method

In soil engineering practice, a *hyperbolic relationship* has been used to represent the stress-strain relationship(Kodner 1963; Duncan and Chang 1970; Sridharan and Narasimha Rao 1980). Kodandaramaswamy and Narasimha Rao (1980) presented a hyperbolic relationship to characterize settlement-time relationship as:

$$s_i = \frac{t_i}{(a_s + b_s t_i)} \tag{5.5}$$

where $s_i=$ settlement at time t_i,

a_s and b_s are constants.

From the property of the rectangular hyperbola, a transformed plot (t_i/s_i vs t_i) is a straight line as seen in Fig.5.2. Equation of the transformed plot is given by:

$$\frac{t_i}{s_i} = a_s + b_s t_i \tag{5.6}$$

and

$$\lim_{t_i \to \infty} s_i = \frac{1}{b_s} \tag{5.7}$$

Hence, the ultimate settlement is equal to the reciprocal of the slope of the straight line given by Eq.5.6.

5.4 Modified Asaoka Method

The best fit line drawn through the n points, $(s_n,\ s_{n-1})$ for n = 1, 2, 3, ... n; is asymptotic to the 45° line (Fig.5.1). So, the point of intersection of the best fit line with the 45° line is not sharp and may change significantly due to even slight shift in the straight line fitted through the plotted points. Hence, different values of s_f can be arrived at for the same set of observed data, while estimating s_f by Asaoka's method.

Asaoka's method is modified as follows:

from Eq.5.2:

$$
\begin{aligned}
& s_{i+1} = \beta_0 + \beta_1 s_i \\
\text{or,}\quad & (s_{i+1} - s_i) = \beta_0 + (\beta_1 - 1)s_i \\
\text{or,}\quad & y_i = \beta_0 + (\beta_1 - 1)s_i \\
\text{where}\quad & y_i = (s_{i+1} - s_i)
\end{aligned}
\tag{5.8}
$$

The straight line representing Eq.5.8 is nearly orthogonal to the s-axis on $(s,\ y)$ plot. Hence, the point of intersection of the straight line fitted through the *(n - 1)*points, $(y_i,\ s_i)$ for i = 1, 2, 3, ...(n-1); with the s-axis is prominent and the value of ultimate settlement can be estimated more accurately.

In this method n settlement values (s_i, i = 1, 2, 3, ...n), at constant intervals of time are observed. Then,

$$y_i = s_{i+1} - s_i, \quad i = 1,\ 2.\ 3,\ ...(n-1)$$

are calculated and (n-1) points $(s_i,\ y_i)$ for i = 1, 2, 3, ... (n-1) are plotted on the (s, y) axes (Fig.5.3). A straight line passing through the points is drawn and extrapolated to intersect s axis. The s value corresponding to (s, 0) represents ultimate settlement (s_f).

5.5 Comparison Of The Three Methods

In the *Asaoka method*, the slope of the line drawn through the plotted points is close to 45°. So to get the ultimate settlement (s_f), the line drawn through the plotted

points is extrapolated and intersected with the 45° line. The intersection point, which represents s_f, may shift significantly due to even a slight shift in the straight line fitted through the plotted points. Hence, different values of s_f can be arrived at for the same set of observed data.

In the case of Modified Asaoka method also, extrapolation is needed. But in this case the problem of intersection of two skew line is avoided, i.e. value of s_f can be determined more accurately. The time-settlement relationships for Asaoka and Modified Asaoka methods are the same, an exponential relationship given by Eq.5.3 or Eq.5.4. The time-settlement relationship is obtained considering a first order approximation of the consolidation equation.

In the case of *hyperbolic plot*, the problem of intersection of two lines is avoided. As, in this case, the time-settlement relationship is assumed to be of second degree, a more accurate value of s_f may be obtained compared to the Asaoka method. Ultimate settlement calculated from the hyperbolic plot will be accurate provided the time-settlement relationship is hyperbolic.

To compare these three methods, the data published by several investigators for a number of field cases have been made use of. For each time-settlement plot s_f has been estimated following all the three methods. Fig.5.4 represents one set of '*settlement vs. time* and *transformed plot of settlement vs. time*' plots. The ultimate settlements predicted by the three observational methods are given in Table-1.

Fig.5.5 represents $(s_f)_{Modified\ Asaoka}$ vs. $(s_f)_{Hyperbolic}$ plot. A best fit straight line has been drawn through all the plotted points. The equation of the straight line is:

$$(s_f)_{Hyperbolic} = 1.136\ (s_f)_{ModifiedAsaoka} \tag{5.9}$$

Fig.5.6 represents $(s_f)_{Modified\ Asoaka}$ vs. $(s_f)_{Asaoka}$ plot. A best fit straight line has been drawn through all the plotted points. The equation of straight line is:

$$(s_f)_{Asaoka} = 1.0\ (s_f)_{ModifiedAsaoka} \tag{5.10}$$

Modified Asaoka and Asaoka methods give the same value of s_f. Modified Asaoka method is suggested to calculate s_f from the observed data, as it avoids finding the intersection between two nearly parallel lines.

5.6 Time Series Analysis

All the points in the transformed time-settlement plots viz. Asaoka, Modified Asaoka and Hyperbolic plots, may not lie on a straight line. In that case, it is to be decided which points are to be considered in estimating ultimate settlement. Long term settlement data published by different authors have been analysed. In estimating ultimate settlement, three sets: (i) first 1/3, (ii) first 2/3 and (iii) last 1/3 observed time-settlement data have been considered in each case. Ultimate settlements have been estimated by: Asaoka, Modified Asaoka and Hyperbolic methods. The values of ultimate settlement estimated by Asaoka Method have been listed in Table-5.2. It is observed that the ultimate settlement estimated using the last 1/3 observed time-settlement data gives a close estimation of the actual value. Similar results have been obtained for Hyperbolic and Modified Asaoka methods. So, only immidiate past settlement observations are to be considered in estimating settlement at subsequent time.

Based on the study presented in this Chapter, the following conclusions can be drawn:

- Ultimate settlement estimated by Hyperbolic method is 1.136 times the corresponding value estimated by Modified Asaoka method.
- Modified Asaoka and Asaoka methods give the same value of ultimate settlement for the set of observed time-settlement data considered.
- Modified Asaoka method is proposed to calculate ultimate settlement from the observed time-settlement data. It is observed that the proposed method gives good estimation of long term settlement.
- Only immediate past settlement observations are to be considered in estimating settlement at subsequent times.

Table 5.1: Comparison of observational methods for prediction of s_f.

Sl. No.	*Reference*		*Ultimate Settlement (cm)* $(s_{i+1} - s_i)$ Vs. s_i Plot cm	t/s Vs. t Plot cm	s_{i-1} Vs. s_i Plot cm
1	Aboshi (1969)	—	203	220.3	212
2	Bhandary (1979)	—	126	178	138
3	Crawford & Sutherland (1979)	—	78.2	82.5	77.5
4	Hansbo (1960)	With Drain (II)	54	62.5	53
		With Drain (III)	69	96.8	70
		No Drain	13	27	13.5
5	Hansbo & Tortenson (1977)	S = 1.6 m	58	70.9	56
		S = 0.8 m	80	90.3	82.5
		No Drain	37	53.3	39.3
6	Havukainen & Vahaaho (1989)	Street & Plaza	20	27.5	20
		Residential Quarter	33	39.2	33
		Industrial Quarter	47	57.5	44.5
		Park Area	62.5	77.5	61
7	Lee et al. (1989)	DS — 2	33.5	38.6	34
		DS — 3	19	26.7	18
		DS — 4	8.9	12.9	8.8
		SP — 23	84	88.9	85
8	Holtz & Broms (1972)	V — 3	61	77.2	60
		V — 8	55	56.7	56
		V — 13	80	87.3	86
		No Sand Drain	38	52.6	37
9	Karunaratne et al. (1989)	S — 41	99	112.5	105
		S — 51	114	136.5	129
10	Kyfor et al. (1988)	S - 1	86.5	94	88
		S - 2	114	124	114
11	Sarkar & Castelli (1988)	SP — 12	99	123	97
		SP — 34	49	60	53
12	Saye et al. (1988)		71	93.3	52.5
13	Sovanic & Widmar (1973)	Tank — $R_2(1)$	66	70.6	71
		Tank — $R_2(5)$	45.5	48	46
		Tank — $R_2(7)$	54.5	57.7	55
14	Tomlinson & Willson (1973)	—	39.6	40.3	40

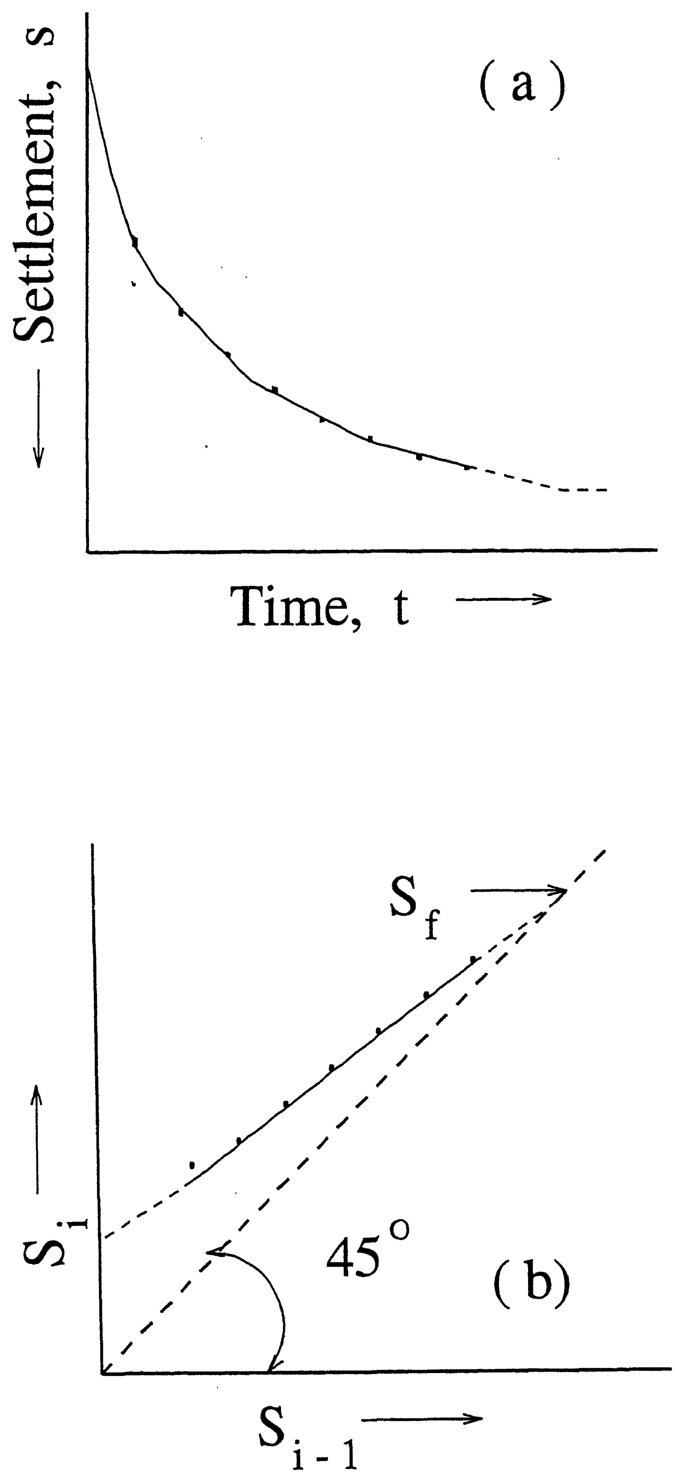


Figure 5.1: Asaoka (1978) method: (a) time-settlement plot, and (b) transformed time-settlement plot to calculate ultimate settlement, s_f.

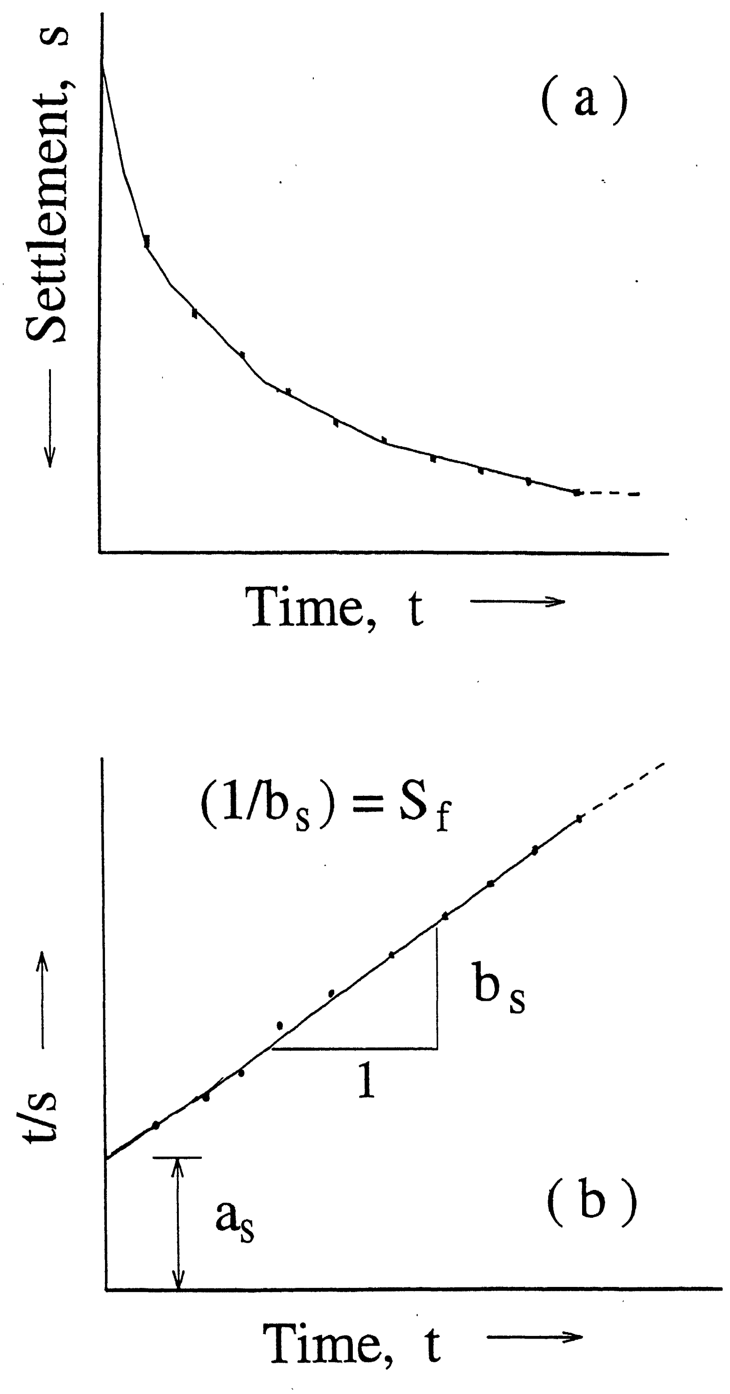


Figure 5.2: Hyperbolic method: (a) time-settlement plot, and (b) transformed time-settlement plot to calculate ultimate settlement, s_f (after Kodandaramaswamy & Narasimha Rao, 1980).

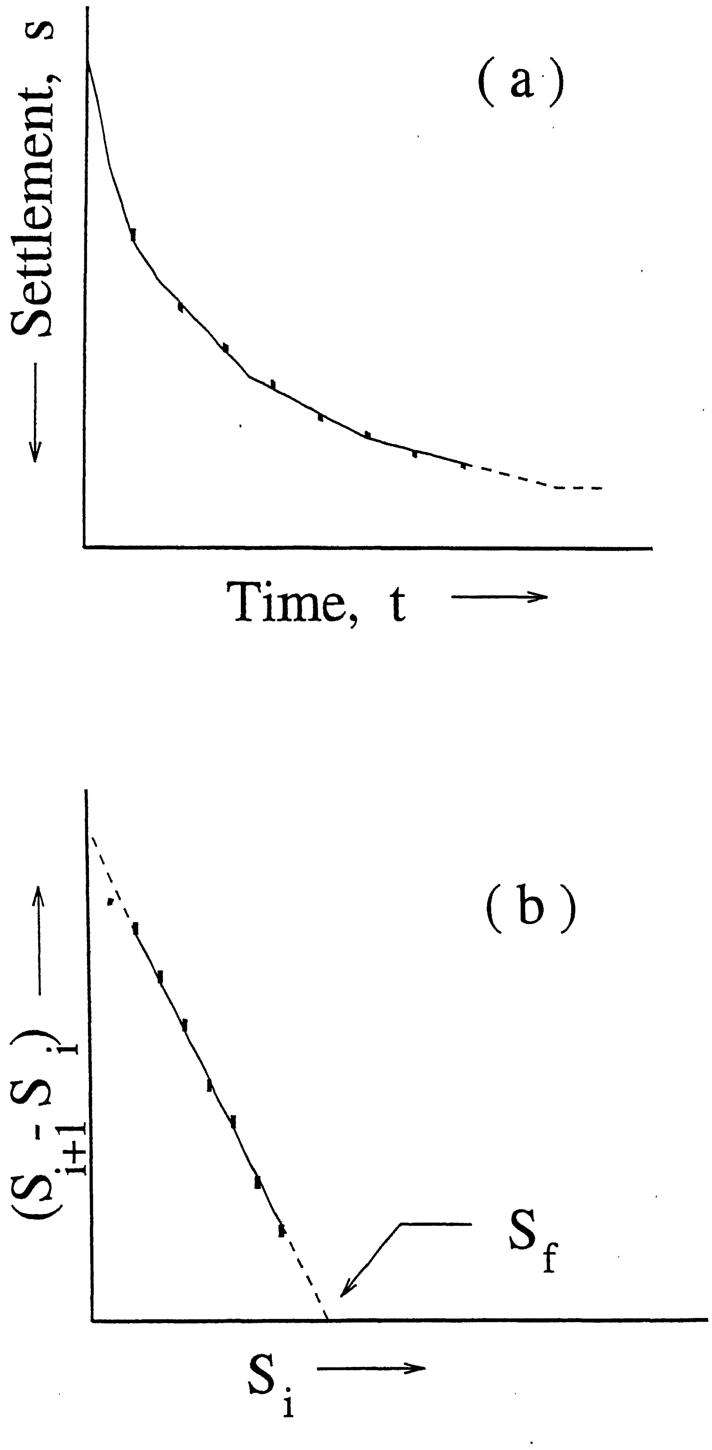
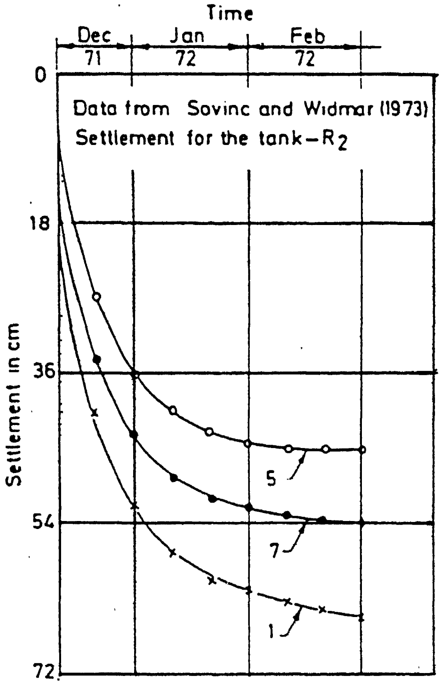
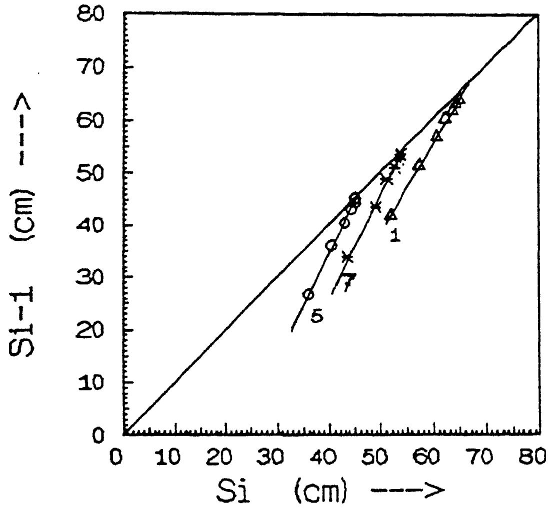


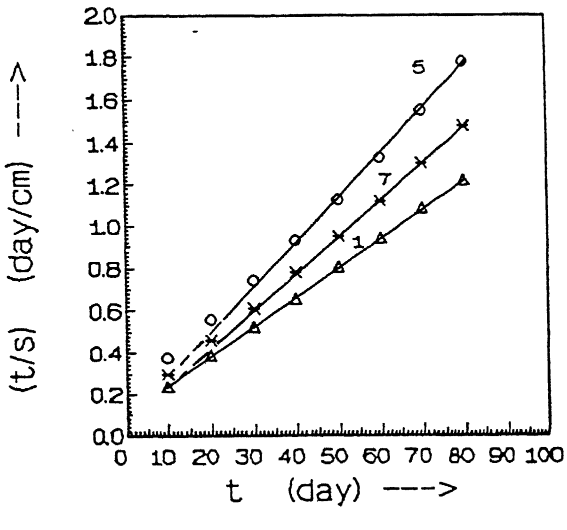
Figure 5.3: Modified Asaoka method: (a) time-settlement plot, and (b) transformed time-settlement plot to calculate ultimate settlement, s_f.



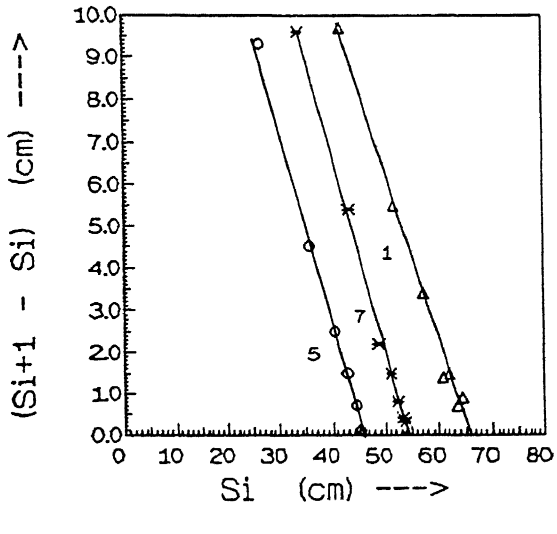
Time vs. Settlement plot



Asaoka plot



Hyperbolic plot



Modified Asaoka plot

Figure 5.4: Time versus settlement plot and it's transformed time-settlement plots to calculate ultimate settlement, s_f.

Table 5.2: Comparison of estimated and measured settlements.

Reference		Estimated Settlement (cm) Considering First 1/3 Points	Estimated Settlement (cm) Considering First 2/3 Points	Estimated Settlement (cm) Considering Last 1/3 Points	Measured Settlement (cm)
Abella, Rivas & Velasco (1988)	C(I)	54.2	52.6	51.7	51.7
	M(I)	75.1	74.0	73.1	70.0
Holtz & Broms (1972)	V-3	65.5	62.3	61	60.0
	V-13	77.3	79.2	80	81.0
Lee et. al (1989)	SP-23	88.0	85.1	84	75.2
Sarkar & Castelli (1988)	SP-12	96.1	98.5	99	100

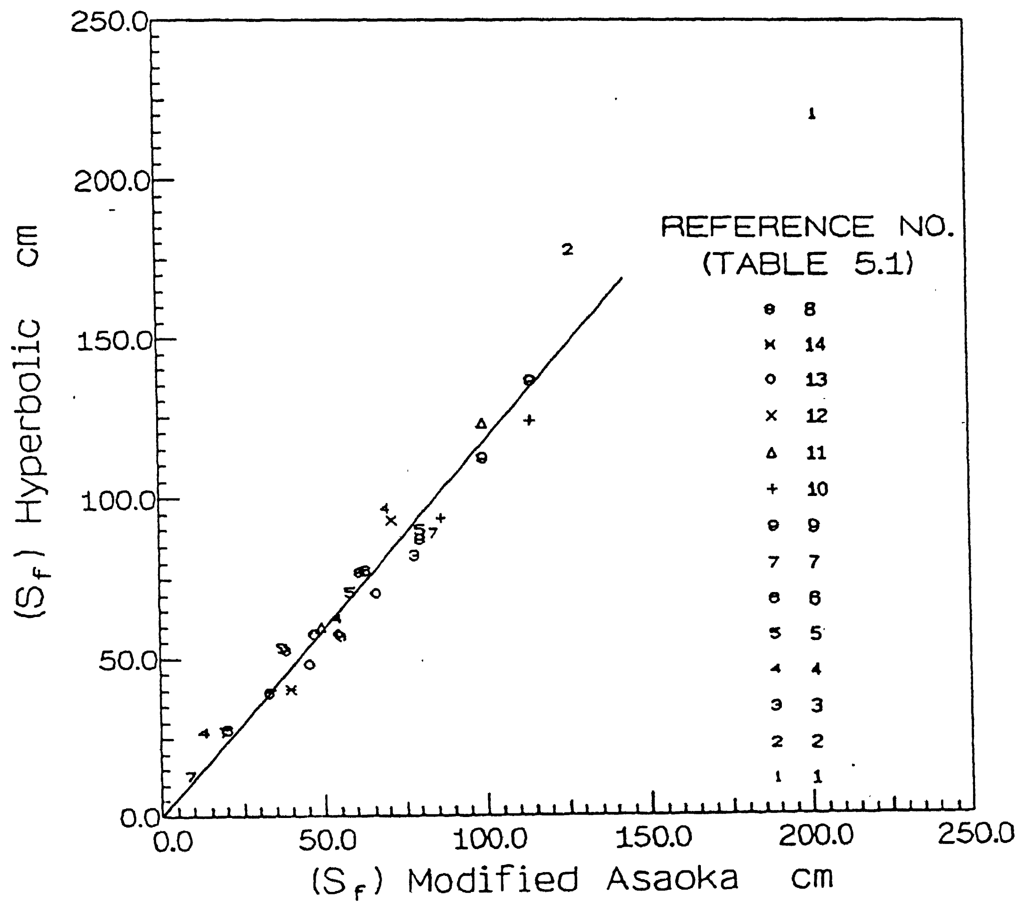


Figure 5.5: Comparison of Modified Asaoka and Hyperbolic methods.

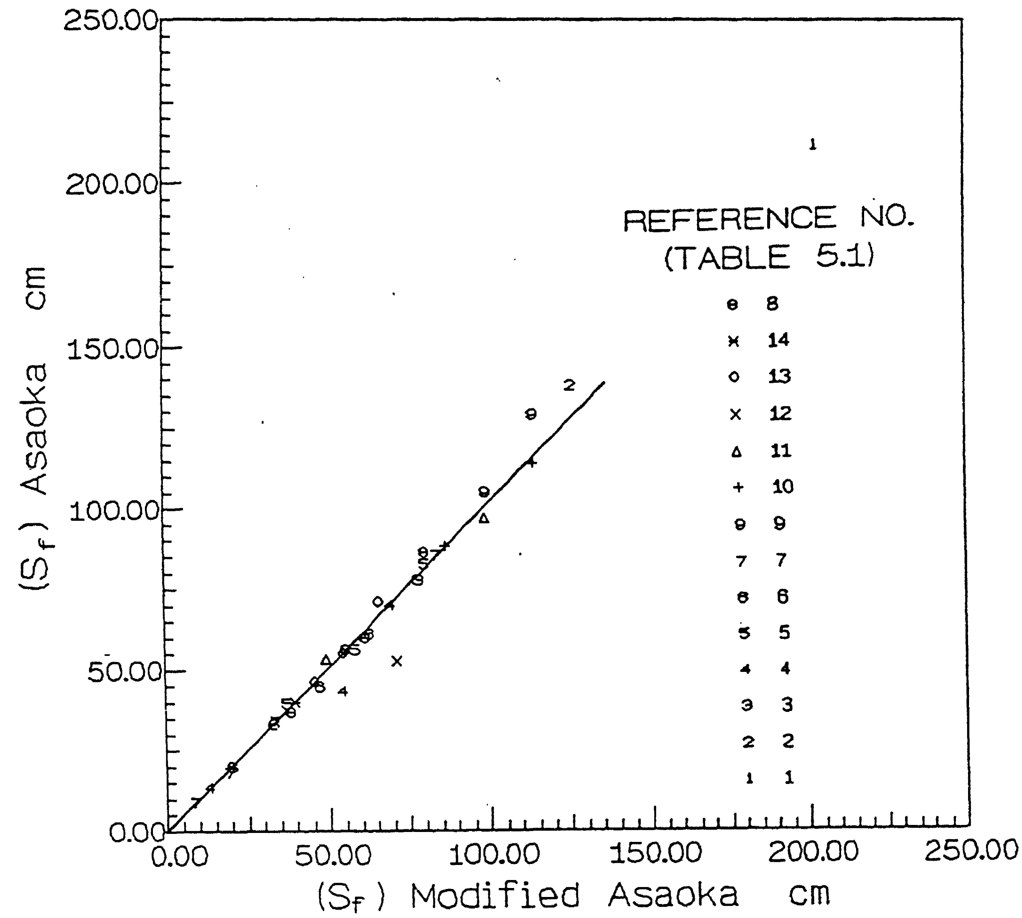


Figure 5.6: Comparison of Modified Asaoka and Asaoka methods.

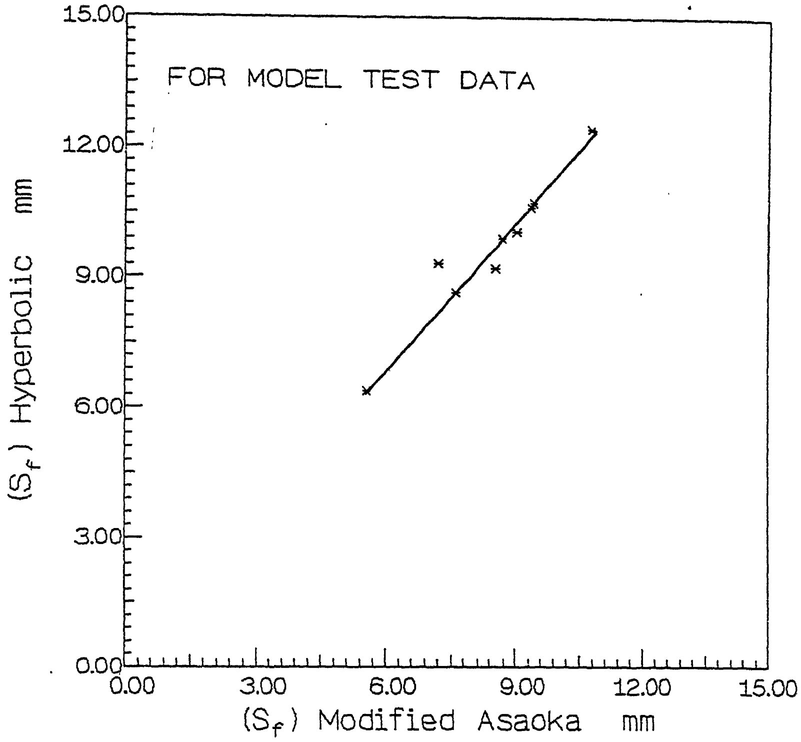
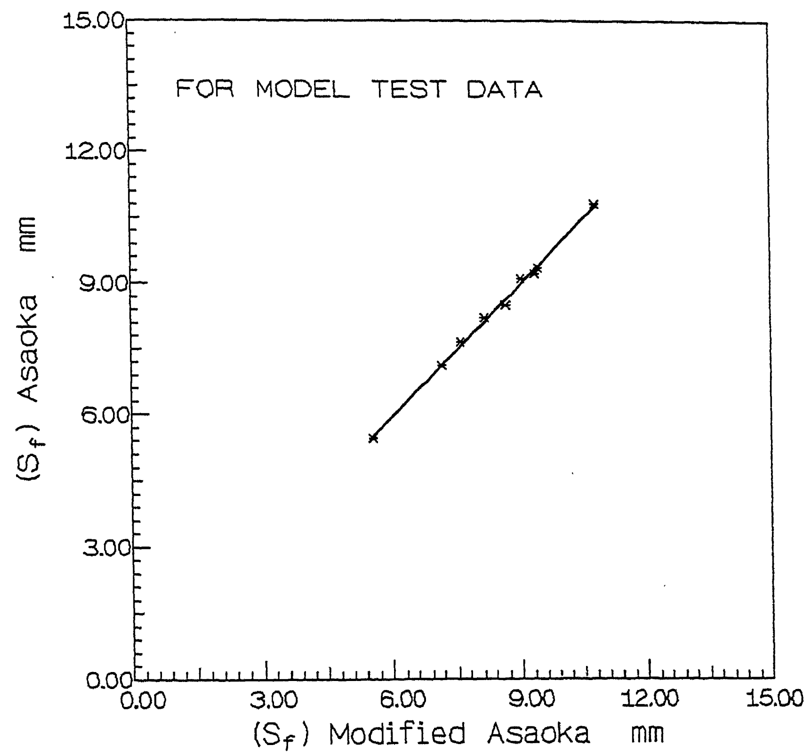


Figure 5.5(a): Comparison of Modified Asaoka and Hyperbolic methods.



Chapter 6

CONCLUDING REMARKS

6.1 Conclusions

Based on the study presented in the previous Chapters, the following conclusions can be drawn:

6.1.1 Model Tests

- For model tests *reconsolidation technique* offers the best choice to prepare small scale, uniform and nearly identical soil deposits in the laboratory with controlled stress history. However, it should be ensured that slurry is well mixed to produce homogeneous deposits.

- Model dimensions and test results are compared with that of the oedometer. The height to diameter ratio of the model is 1.667 which is five times the height to diameter ratio of oedometer, 0.333. In the case of model test also, lubricating oil was applied on the inner surface of the tank to reduce side friction. Final settlement observed in the model tests are on an average 84.3% of the estimated settlements based on m_v values from the oedometer test. The C_{vz} values estimated from the model test data are approximately 95.5% of the corresponding values from the oedometer test. Compression index (C_c) obtained from model test is 0.224, which is slightly less than that from oedometer test, 0.233.

- In the model tests, silty clay was used. So, during drain installation soil may be partly densified. In some cases ultimate settlement values obtained from model tests with drain installation are smaller than the corresponding values for untreated sample. Due to partial densification during drain installation less settlement may have occured in those cases. Possibility of soil getting densified in the field, due to vertical drain installation is very less, as vertical drains are often installed in the saturated clay. In the model test the two phenomenon: (i)

densification and (ii) disturbance occured while in the field mainly disturbance occurs.

- It is observed that disturbance due to drain installation increases with the increase in size of shoe, OCR and aging. This conclussion is arrived at from the analysis of time-settlement data, cone penetration resistances and water content measurements.

6.1.2 Estimation Of Long Term Settlement

- Asaoka's and hyperbolic methods to estimate long term settlement from observed time-settlement data are discussed. A new observational method to estimate long term settlement is proposed modifying Asaoka's method, and compared with the Asaoka's and hyperbolic methods. It is observed that the proposed method gives good estimation of long term settlement.

- Only immediate past settlement observations are to be considered in estimating settlement at subsequent time.

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